EXPERIMENTAL STUDY ON
SHEAR STRENGTHENING OF RC T-BEAMS
WITH WEB OPENINGS USING FRP COMPOSITES

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A THESIS SUBMITTED IN PARTIAL FULFILMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

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in

Structural Engineering

by

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CERTIFICATE

*This is to certify that the thesis entitled, **"EXPERIMENTAL STUDY ON SHEAR STRENGTHENING OF RC T-BEAMS WITH WEB OPENINGS USING FRP COMPOSITES"** submitted by **SREELATHA VUGGUMUDI** bearing Roll No. **211CE2032** in partial fulfilment of the requirements for the award of **Master of Technology** Degree in **Civil Engineering** with specialization in **"Structural Engineering"** during 2012-13 session at National Institute of Technology, Rourkela is an authentic work carried out by her under our supervision and guidance.*

To the best of our knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

Prof. K. C. Biswal

Date:
Place:Rourkela

ABSTRACT

Shear collapse of reinforced concrete (RC) members is catastrophic and occurs suddenly with no advance warning of distress. In several occasions existing RC beams have been found to be deficient in shear and in need of strengthening. Conventional shear strengthening method such as external post tensioning, member enlargement along with internal transverse steel, and bonded steel plates are very costly, requiring extensive equipment, time, and significant labor. Conversely, the relatively new alternative strengthening technique using advanced composite materials, known as fiber reinforced polymer (FRP), offers significant advantages such as flexibility in design, ease of installation, reduced construction time, and improved durability.

The overall objective of this study was to investigate the shear performance and failure modes of RC T-beams strengthened with externally bonded GFRP sheets. In order to achieve these objectives, an extensive experimental program consisting of testing eleven, full scale RC beams was carried out. The variables investigated in this study included steel stirrups, shear span-to-depth ratio, GFRP amount.

The experimental results indicated that the contribution of externally bonded GFRP to the shear capacity is significant and depends on the variable investigated. The failures of strengthened beams are initiated with the debonding failure of FRP sheets followed by brittle shear failure. However, the shear capacity of these beams has increased as compared to the control beam which can be further improved if the debonding failure is prevented. An innovative method of anchorage technique by using GFRP plates has been used to prevent these premature failures, which as a result ensure full utilization of the strength of FRP. A theoretical study is also proposed by using ACI guidelines for computing the shear capacity of the strengthened beams.

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The satisfaction and euphoria on the successful completion of any task would be incomplete without the mention of the people who made it possible whose constant guidance and encouragement crowned out effort with success.

I would like to express my heartfelt gratitude to my esteemed supervisor, Prof. Kishore Chandra Biswal for his technical guidance, valuable suggestions, and encouragement throughout the experimental and theoretical study and in preparing this thesis. It has been an honour to work under Prof. K.C.Biswal , whose expertise and discernment were key in the completion of this project.

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NOTATIONS

A_{st}	Area of steel
a	shear span
b_f	width of the flange
b_w	width of the web
d_f	depth of the flange
d_w	depth of the web
d	effective depth
d’	effective cover
D	Overall depth of the beam
ρ	reinforcing ratio
ρ_{max}	maximum reinforcing ratio
φ	diameter of the reinforcement
f_y	yield stress of the reinforcement bar
L	span length of the beam
P_u	ultimate load
λ	load enhancement ratio

ACRONYMS AND ABBREVATIONS

ACI	American Concrete Institute
CBA	Control Beam of Group-A
CBB	Control Beam of Group-B
CFRP	Carbon Fiber Reinforced Polymer
EB	Externally Bonded
FRP	Fiber Reinforced Polymer
FGPB	Fiber Glass Plate Bonding
GFRP	Glass Fiber Reinforced Polymer
HYSD	High-Yield Strength Deformed
IS	Indian Standard
NSM	Near Surface Mounted
PSC	Portland Slag Cement
RC	Reinforced Concrete
SBA2	Strengthened Beam of Group-A strengthened with 2-L-GFRP
SBA4	Strengthened Beam of Group-A strengthened with 4-L-GFRP
SBB2	Strengthened Beam of Group-B strengthened with 2-L-GFRP

CHAPTER – 1

INTRODUCTION

CHAPTER - 1

INTRODUCTION

1.1 PREAMBLE

Many natural disasters, earthquake being the most affecting of all, have produced a need to increase the present safety levels in buildings. The knowledge of understanding of the earthquakes is increasing day by day and therefore the seismic demands imposed on the structures need to be revised. The design methodologies are also changing with the growing research in the area of seismic engineering. So the existing structures may not qualify to the current requirements. As the complete replacement of such deficient structures leads to incurring a huge amount of public money and time, retrofitting has become the acceptable way of improving their load carrying capacity and extending their service lives.

Retrofitting is specially used to relate to the seismic upgrade of facilities, such as in the case of the use of composite jackets for the confinement of columns. Retrofitting is making changes to an existing building to protect it from flooding or other hazards such as high winds and earthquakes.

The maintenance, rehabilitation and upgrading of structural members, is perhaps one of the most crucial problems in civil engineering applications. Moreover, a large number of structures constructed in the past using the older design codes in different parts of the world are structurally unsafe according to the new design codes. Since replacement of such deficient elements of structures incurs a huge amount of public amount and time, strengthening has become the acceptable way of improving their load carrying capacity and extending their service lives.

To meet up the requirements of advance infrastructure new innovative materials/ technologies in civil engineering industry has started to make its way. Any technology or material has its limitations and to meet the new requirements new technologies have to be invented and used. With structures becoming old and the increasing bar for the constructed buildings the old buildings have started to show a serious need of additional retrofits to increase their durability and life.

The retrofitting is one of the best options to make an existing inadequate building safe against future probable earthquake or other environmental forces. There are many other factors, considered in decision making for any retrofitting strategy.

This proves to be a better option catering to the economic considerations and immediate shelter problems rather than replacement of buildings. Because replacement is very costly and structural behaviour also may change and it may cause inconvenience also.

There are several situations in which a civil structure would require retrofitting or rehabilitation. The following are some reasons that may need retrofitting

1. Building which are designed considering gravity loads only.
2. Development activities in the field of Earthquake Resistant Design (EQRD) of buildings and other structures result into change in design concepts.
3. Lack of timely revisions of codes of practice and standards.
4. Lack of revisions in seismic zone map of country.
5. In cases of alterations in buildings in seismic prone area i.e. increase in number of story, increase in loading class etc.

6. In cases of deterioration of Earthquake (EQ) forces resistant level of building e.g. decrease in strength of construction material due to decay, fire damage, and settlement of foundations.

7. The quality of construction actually achieved may be lower than what was originally planned.

8. Lack of understanding by the designer.

9. Improper planning and mass distribution on floors.

Techniques

Generally four characteristics are defining a structure: load carrying capacity, durability, functionality and aesthetics. From these the first three are considered the most important, mainly for safety and comfort reasons. When one of these functions is not fulfilled, a construction may be in need of:

- maintenance – keep the structure at a desired performance level, e.g. a steel bridge has to be periodically painted to avoid corrosion
- repair – upgrade the structure to its original design level, e.g. a structure damaged from an earthquake has to be structurally repaired to be at the same performance level as before the earthquake
- upgrading – to increase the performance of a structure to a higher level, e.g. if the traffic load on a bridge has increased, so the load bearing capacity has to be increased

The actions mentioned above, are called rehabilitation methods.

There are mainly two types of techniques are available

1. Seismic resistance based design
    - Concrete jacketing
    - Steel jacketing
    - FRP wrapping
2. Seismic response control design
    - Elastic-plastic dampers
    - Base isolators
    - Tuned liquid dampers

And also many options for retrofitting a structure are possible

The ones which are used traditionally for a long time now such as addition of new Shear walls, addition of infill walls, addition of wing (side) walls, addition of buttresses, jacketing of reinforced concrete members, propping up , steel collars, casing, building up, bonding steel plates or steel jacketing. However, with increase in research and introduction of new materials and technology there are new ways of retrofitting the structure with many added advantages. Introduction of Fibre Reinforced Composites being one of them. It has proved to be a promising material and technology in repairs and retrofitting.

Fibre Reinforced Polymer (FRP):

Fibre Reinforced Polymer (FRP) composites comprise fibres of high tensile strength within a polymer matrix such as vinylester or epoxy. FRP composites have emerged from being exotic materials used only in niche applications following the Second World War, to common engineering materials used in a diverse range of applications such as aircraft, helicopters, space-craft, satellites, ships, submarines, automobiles, chemical processing equipment, sporting goods and civil infrastructure. The role of FRP for strengthening of existing or new reinforced concrete structures is growing at an extremely rapid pace owing mainly to the ease and speed of construction, and the possibility of application without disturbing the existing functionality of the structure. FRP composites have proved to be extremely useful for strengthening of RCC structures against both normal and seismic loads.

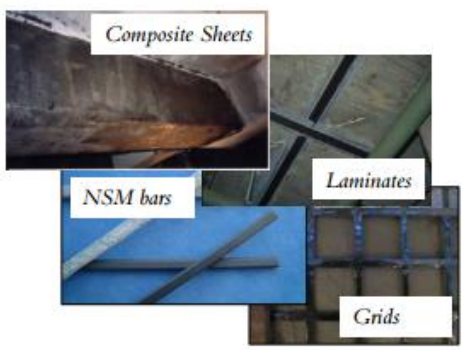


Figure 1-1. Various composite materials

Advantages over other materials:

1.FRP is corrosion proof. When steel is in contact with water, oxygen, or other strong oxidants, or acids, it rusts.

2. Easy in transportation, can be easily rolled.

3. High fatigue resistance.

4. Light weight. Hence, very high strength to weight ratio. The lower weight makes handling and installation significantly easier than steel. This is particularly important when installing material in cramped Locations.

5. Fiber composite materials are available in very long lengths while steel plate is generally limited to 6m. The availability of long length and the flexibility of the material also simplifies installation and joints and laps are also not required.

6. Very less period of time is required.

7. Does not impact on detailing or form of historic structures. In general for FRP rapping no bolts are required, In fact use of bolts would seriously weaken the material unless additional cover plates are bonded on. Furthermore, because there is no need to drill into the structure to fix the bolts or other mechanical anchors there is no risk of damaging existing reinforcement

8. Low unit weight (150-900 g /m^2).

9. Fiber composite strengthening materials have higher ultimate strength and lower density than steel.

10. low energy consumption during fabrication of raw material and structure, and the potential for real time monitoring.

Disadvantages:

1.The main disadvantage of externally strengthening structures with composite materials is the risk of fire, vandalism or accidental damage, unless the strengthening is protected.

2. Below 5˚c temperature we cannot use FRP.

3. The lack of experience of the techniques and suitably qualified staff to carry out the work.

4. Lack of accepted design standards.

Strengthening using FRP:

Most of the elements of a structure can be strengthened with FRP composite materials. This means in fact that FRP composites can take up the majority of the forces developed in a structure as long as they are transmitted by the strengthened element to the composite as tensile stresses.

Strengthening with externally bonded FRP fabric has shown to be applicable to many kinds of structures. Currently, this method has been applied to strengthen such structures as column, beams, walls, slabs, etc.

The use of external FRP reinforcement may be classified as

- flexural strengthening
- improving the ductility of compression members
- shear strengthening.

Flexural strengthening using FRP:

The laminates are generally made up of Carbon fibres blended in an epoxy matrix. These when applied with epoxy, act as external tension reinforcements to increase the flexural strength of the RCC members.

Beams, Plates and columns may be strengthened in flexure through the use of FRP composites bonded to their tension zone using epoxy as a common adhesive for this purpose. The direction of fibers is parallel to that of high tensile stresses. Both prefabricated FRP strips, as well as sheets (wet-lay up) are applied.

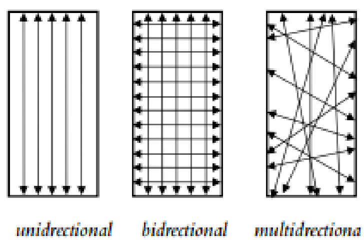


Figure 1-2. Fibre directions in composite materials

Shear strengthening:

When the RC beam is deficient in shear, or when its shear capacity is less than the flexural capacity after flexural strengthening, shear strengthening must be considered. It is critically important to examine the shear capacity of RC beams which are intended to be strengthened in flexure.

Various FRP bonding schemes have been used to increase the shear resistance of RC beams. These includes bonding FRP to the sides of the beam only, bonding FRP U jackets to both the sides and the tension face, and wrapping FRP around the whole cross section of the beam. The use of fibres in two directions can obviously be beneficial with respect to shear resistance even if strengthening for reversed loading is not required, except for unlikely case in which one of the fibre directions is exactly parallel to the shear cracks. In this sense, FRP plates with fibres in three or more directions may also be used.

FRPs are strong only in the directions of fibres, the fibres may be oriented in such directions as to control shear cracks. Because shear forces and bending moments in a beam may be reversed under conditions such as cyclic loading and earthquake attacks, fibres may thus be arranged at two different directions to satisfy the requirement of shear strengthening in both directions.

Two main modes of failure are recognized by the research society for FRP strengthened beams are:

Fibre failure in the FRP:

It occurs when the tensile stress in the fibres exceeds the tensile strength. It is characterized by a rapid progressive fibre failure in the composite, especially for sheets, but the failure is in most of the cases brittle. The orientation of the fibres with respect to the principal strain in concrete affects the ductility of the composite.

Anchorage failure :

Also known as bond failure is governed by the properties of the weakest materials in contact, i.e. concrete and adhesive. When the shear strength of one of these two exceeds the force then transfer cannot be ensured anymore and a “slip” is produced. The debonding can take place in concrete, between the concrete and adhesive, in the adhesive, between the adhesive and the fibres. The most common debonding failure observed is at the surface of the concrete, which is an explicable phenomenon since the concrete is the weakest element in this “interaction chain”. The anchorage failure is considered as more dangerous than tensile failure because it cannot be foreseen and can almost not be controlled at all.

The failure mode of a strengthened beam depends also on the configuration of the strengthening used. From the configurations used the one to be avoided is the side bonded because it is the most exposed to debonding failure due to its limited anchorage length. The fully wrapped configuration is the safest since is the failure is controlled by fibre rupture, but it is quite uncommon with this type of free bound configuration for a beam in a structure. Probably the most used configuration is the U wrapped system. Since the beams are connected to slabs, consequently T section behaviour, this configuration is safer than the side bonding but still has critical regions.

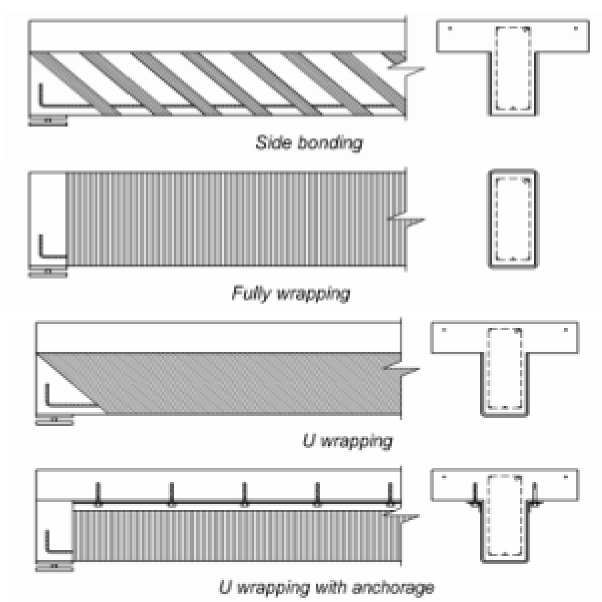


Figure 1-3. Shear strengthening configurations using FRP

Anchorage devices for FRP reinforcement used to strengthen members in flexure

Three general categories of anchorage type have been investigated to date to prevent debonding in RC members strengthened with FRP

(a) U-jacket anchors

(b) Mechanically fastened metallic anchors

(c) FRP anchors

FRP U-jacket anchors:

FRP U-jacket anchors involve the application of uni-directional or bi-directional fibre to the ends of flexural FRP reinforcement to prevent or delay debonding initiating from the plate end. U jackets can also be placed along the length of the member to prevent or delay debonding initiating away from the plate end. The ultimate function of a U-jacket is to provide the confinement necessary to resist the tensile peeling stresses and longitudinal crack propagation at fibre termination points or intermediate cracks.

Metallic Anchorage Systems: Metallic anchorages are one of the earliest forms of FRP end anchorage devices investigated by researchers (e.g. Sharif et al. 1994; Jensen et al. 1999). Investigations have been conducted on:

adhesively bonded metallic plates with mechanical fasteners (refer Figure 4), adhesively bonded metallic U-jackets, and those with end clamping . Researchers have found the use of metallic anchorages to provide a superior increase in anchorage strength in addition to ductility enhancement.

FRP Anchors:

Anchors made from rolled fibre sheets or bundled loose fibres are a promising form of anchorage because they can be applied to wide FRP-strengthened structural elements such as slabs and walls. They are discrete and do not suffer from the same constraints as U-jackets. Such anchors are referred to as FRP spike anchors, fibre anchors, fibre bolts and FRP dowels, amongst other names, but are herein collectively referred to FRP anchors. The anchor can be hand-made (in the laboratory or on site) or manufactured from glass or carbon fibre sheets or loose fibres which have been rolled or bundled.

Anchorage devices used for FRP reinforcement

Although fully wrapping the beam cross-section with FRP has been demonstrated to provide the most effective strengthening solution for shear and torsion applications. It is seldom achieved in practice due to the presence of physical obstructions such as beam flanges. This form of failure is usually premature, sudden, non-ductile and has resulted in the development of many innovative anchorage details at the web-flange interface. These include

- FRP enveloping the web of the beam in a U-shape, including termination at the underside of the beam flange with no anchorage.
- Wrapping the web and flange of the beams through drilled holes through the beam flanges.
- Mechanically fastened metallic anchors installed at the underside of the beam flange to anchor FRP U wrap legs.
- Embedment of the FRP U-jacket legs into the beam flanges, through pre-cut grooves using adhesive bonding.

- FRP anchors installed to restrain the legs of the FRP U-jackets.

- Mechanical substrate strengthening over the anchorage zone of FRP shear reinforcement

- FRP enveloping the web of the beam and anchored at the underside of the beam flanges with unidirectional or bi-directional fibres.

U jacketing is now the most popular shear strengthening solution due to its high practicality, but it is limited by end peeling of the U-jacket legs.

1.2OBJECTIVE

The main objectives of the present work are:

- To study the structural behaviour of reinforced concrete (RC) T-beams with a transverse hole under static loading condition.

- To study the contribution of externally bonded Fiber Reinforced Polymer (FRP) sheets on the shear behaviour of RC T-beams.

- To know the suitability of the FRP composites as repair materials for deteriorated RC Structures.
- To examine the effect of different parameters such as steel stirrups, number of layers, different shear span to effective depth ratio etc. on enhancement of load carrying capacity and load deflection behaviour.

- To investigate the effect of a new anchorage scheme on the shear capacity of the beam.

1.3 THESIS ORGANIZATION

The present thesis is divided into six chapters.

The general introduction to retrofitting of reinforced concrete (RC) beams and its importance in different engineering fields along with the objective of the present work are outlined in chapter 1.

A review comprising of literature on strengthening of different types of beams under different load, support conditions and different orientation of fiber are presented in chapter 2. The critical observations on earlier published works are highlighted and the scope of the present research work is outlined.

Chapter 3 deals with the description of the experimental program. The constituent materials, the beam specimens, and FRP installation procedure are presented. A brief description of test set up and procedure is given.

Chapter 4 contains the test results and discussion. The observed crack behaviours and modes of failure are reported. In addition, comparisons among test results are given.

Chapter 5 deals with the design approach for computing the shear capacity of the strengthened beams.

The important conclusions and the scope for further extension of the present work are outlined in chapter 6.

A list of important references cited in the present thesis is presented at the end.

CHAPTER – 2

REVIEW OF LITERATURE

CHAPTER - 2

REVIEW OF LITERATURE

2.1 BRIEF REVIEW

The state of deterioration of the existing civil engineering concrete structures is one of the greatest concerns to the structural engineers worldwide. The renewal strategies applied to existing structures comprise of rehabilitation and complete replacement. The latter involves a huge expenditure and time; hence the rehabilitation is the only option available. Fiber reinforced polymers (FRP) are the promising materials in rehabilitation of the existing structures and strengthening of the new civil engineering structures.

This chapter presents a brief review of the existing literature in the area of reinforced concrete (RC) beams strengthened with epoxy-bonded FRP. The major achievements and results reported in the literature are highlighted. The review of the literature is presented in the following three groups:

a) Strengthening of Reinforced Concrete (RC) Rectangular Beams

b) Strengthening of Reinforced Concrete (RC) T-Beams

c) Strengthening of RC Rectangular and T- Beams with web opening

2.2 Strengthening of Reinforced Concrete (RC) Rectangular Beams:

Ghazi et al. (1994) studied the shear repair of reinforced concrete (RC) beams strengthened with fiber glass plate bonding (FGPB) for structural and non-structural cracking behaviour due to a variety of reasons. Results from a study on strengthening of RC beams having deficient shear strength and showing major diagonal tension cracks have been presented. The beams with deficient shear strength were damaged to a predetermined level (the appearance of the first shear crack) and then repaired by fiber glass plate bonding (FGPB) techniques. Different shear repair schemes using FGPB to upgrade the beams shear capacity were used, i.e., FGPB repair by shear strips, by shear wings, and by U-jackets in the shear span of the beams. The study results also show that the increase in shear capacity by FGPB was almost identical for both strip and wing shear repairs. However, this increase was not adequate to cause beams repaired by these two schemes to fail in flexure.

Chaallal et al. (1998) investigated a comprehensive design approach for reinforced concrete flexural beams and unidirectional slabs strengthened with externally bonded fiber reinforced plastic (FRP) plates. The approach complied with the Canadian Concrete Standard. This was divided into two parts, namely flexural strengthening and shear strengthening. In the first part, analytical models were presented for two families of failure modes: classical modes such as crushing of concrete in compression and tensile failure of the laminate, and premature modes such as debonding of the plate and ripping off of the concrete cover. These models were based on the common principles of compatibility of deformations and equilibrium of forces. In the second part, design equations were derived to enable calculation of the required cross-sectional area of shear lateral FRP plates or strips for four number of plating patterns: vertical strips, inclined strips, wings, and U-sheet jackets.

Khalifa et al. (2000) studied the shear performance and the modes of failure of reinforced concrete (RC) beams strengthened with externally bonded carbon fiber reinforced polymer (CFRP) wraps experimentally. The experimental program consisted of testing twenty-seven, full-scale, RC beams. The variables investigated in this research study included steel stirrups (i.e., beams with and without steel stirrups), shear span-to depth ratio (i.e., a/d ratio 3 versus 4), CFRP amount and distribution (i.e., Continuous wrap versus strips), bonded surface (i.e., lateral sides versus U-wrap), fiber orientation (i.e., 90°/0° fiber combination versus 90° direction), and end anchor (i.e., U-wrap with and without end anchor). The experimental results indicated that the contribution of externally bonded CFRP to the shear capacity is significant and dependent upon the variable investigated. For all beams, results show that an increase in shear strength of 22 to 145% was achieved.

Alex et al. (2001) studied experimentally the effect of shear strengthening of RC beams on the stress distribution, initial cracks, crack propagation, and ultimate strength. Five types of beams with different strengthening carbon-fiber–reinforced plastic sheets are often strengthened in flexure. The experimental results show that it is not necessary to strengthen the entire concrete beam surface. The general and regional behaviors of concrete beams with bonded carbon-fiber–reinforced plastic sheets are studied with the help of strain gauges. The appearance of the first cracks and the crack propagation in the structure up to the failure is monitored and discussed for five different strengthened beams. In particular, for one of the strengthened RC beams, the failure mode and the failure mechanism are fully analyzed.

Sheikh (2002) studied on retrofitting with fiber reinforced polymers (FRP) to strengthen and repair damaged structures, which was a relatively new technique. In an extensive research programme at the University of Toronto, application of FRP in concrete structures was being investigated for its effectiveness in enhancing structural performance both in terms of strength and ductility. The structural components tested so far include slabs, beams, columns and bridge culverts. Research on columns had particularly focused on improving their seismic resistance by confining them with FRP. All the specimens tested were considered as full-scale to two-third scale models of the structural components generally used in practice. Results indicated that retrofitting with FRP offers an attractive alternative to the traditional techniques.

Chen and Teng (2003) carried out an investigation on the shear capacity of FRP-strengthened RC beams. These studies have established clearly that such strengthened beams fail in shear mainly in one of the two modes, i.e., FRP rupture and FRP debonding, and have led to preliminary design proposals. This study was concerned with the development of a simple, accurate and rational design proposal for the shear capacity of FRP-strengthened beams which fail by FRP debonding. This new model explicitly recognises the non-uniform stress distribution in the FRP along a shear crack as determined by the bond strength between the FRP strips and the concrete.

2.3 Strengthening of Reinforced Concrete (RC) T-Beams :

Hamid et al. (1992) have investigated the static strength of reinforced concrete beams strengthened by gluing glass-fiber-reinforced-plastic (GFRP) plates to their tension flanges experimentally. Five rectangular beams and one T-beam were tested to failure under four-point bendingThe results indicate that the flexural strength of RC beams can be significantly increased by gluing GFRP plates to the tension face. In addition, the epoxy bonded plates improved the cracking behavior of the beams by delaying the formation of visible cracks and reducing crack widths at higher load levels.

Sayed et al. (1999) have investigated the behavior of reinforced concrete beams strengthened with various types of fiber reinforced polymer (FRP) laminates. The ratio of absorbed energy at failure to total energy, or energy ratio, was used as a measure of beam ductility. It is concluded that the presence of vertical FRP sheets along the entire span length eliminates the potential for rupture of the longitudinal sheets. The combination of vertical and horizontal sheets, together with a proper epoxy, can lead to a doubling of the ultimate load carrying capacity of the beam. However, all the strengthened beams experienced brittle failure, mandating a higher factor of safety in design.

Khalifa et al. (2000) has investigated the shear performance of reinforced concrete (RC) beams with T-section. The experimental program consisted of six full-scale, simply supported beams. The parameters investigated in this study included wrapping schemes, CFRP amount, 90°/0° ply combination, and CFRP end anchorage. The experimental results show that externally bonded CFRP can increase the shear capacity of the beam significantly. In addition, the results indicated that the most effective configuration was the U-wrap with end anchorageResults showed that the proposed design approach is conservative and acceptable.

Khalifa et al. (2000) has investigated the shear performance and the modes of failure of reinforced concrete (RC) beams strengthened with externally bonded carbon fiber reinforced polymer (CFRP) wraps. The experimental program consisted of testing twenty-seven, full-scale, RC beams. As part of the research program, the experimental study examined the effectiveness of CFRP reinforcement in enhancing the shear capacity of RC beams in negative and positive moment regions, and for beams with rectangular and T-cross section. The experimental results indicated that the contribution of externally bonded CFRP to the shear capacity is significant and dependent upon the variable investigated.

Triantafillou et al.(2000) has investigated a simple design model for the calculation of the fiber reinforced polymer (FRP) contribution to the shear capacity of strengthened RC elements according to the design formats of the Eurocode, American Concrete Institute, and Japan Concrete Institute. The key element in the model is the calculation of an effective FRP strain, which is calculated when the element reaches its shear capacity due to concrete diagonal tension. Diagonal tension failure may be combined with FRP debonding or tensile fracture, and the latter also may occur at a stage beyond the ultimate shear capacity. Finally it is demonstrated that, when compared with others, the proposed model gives better agreement with most of the test results available.

Ozgur Anil (2008) have studied various methods are developed for strengthening reinforced concrete beams against shear. This study presents test results on strengthening of shear deficient RC beams by external bonding of carbon fiber reinforced polymer (CFRP) straps. Six RC beams with a T-section were tested under cyclic loading in the experimental program. Shear deficient beams with low strength concrete were strengthened by using CFRP straps for obtaining ductile flexural behavior. The test results confirmed that all CFRP arrangements improved the strength, stiffness and energy dissipation capacity of the specimens significantly. The failure mode and ductility of specimens were proved to differ according to the CFRP strap width and arrangementalong the beam.

Khaled et al. (2009) have studied the feasibility and effectiveness of a new method of strengthening existing RC T-Beams in shear by using mechanically anchored unbounded dry carbon fiber (CF) sheets. This method eliminates the debonding of epoxy-bonded carbon-fiber-reinforced polymer (CFRP) sheets and utilizes the fully capacity of dry sheets. In this method, dry CF sheets are wrapped around and bonded to two steel rods. Then the rods are anchored to the corners of the web-flange intersection of the T-beam with mechanical bolts. The test results showed that the beam strengthened by the new mechanically anchored dry CF had about 48%

increase in shear capacity as compared to the control beam and 16% increase in shear capacity as compared to the beam strengthened by CFRP epoxy-bonding method.

Sundarraja et al. have studied a number of studies on shear strengthening of RC beams using externally bonded fiber reinforced polymer sheets, the behavior of FRP strengthened beams in shear is not fully understood. The objective of this study is to clarify the role of glass fiber reinforced polymer inclined strips epoxy bonded to the beam web for shear strengthening of reinforced concrete beams. The study also aims to understand the shear contribution of concrete, shear strength due to steel bars and steel stirrups and the additional shear capacity due to glass fiber reinforced polymer strips in a RC beam. And also to study the failure modes, shear strengthening effect on ultimate force and load deflection behavior of RC beams bonded externally with GFRP inclined strips on the shear region of the beam.

Tanarslan et al. (2009) have studied an experimental investigation on T- section reinforced concrete (RC) beams strengthened with externally bonded carbon fiber-reinforced polymer (CFRP) strips. Five shear deficient specimens were strengthened with side bonded and U-jacketed CFRP strips, remaining one tested with its virgin condition without strengthening. The main objective was to analyze the behavior and failure modes of T-section RC beams strengthened in shear with externally bonded CFRP strips. According to test results premature debonding was the dominant failure mode of externally strengthened RC beams so the effect of anchorage usage on behavior and strength was also investigate.

Heyden et al.(2010) have investigate the results of an experimental study to the behavior of structurally damaged full-scale reinforced concrete beams retrofitted with CFRP laminates in shear or in flexure. The experimental results, generally, indicate that beams retrofitted in shear and flexure by using CFRP laminates are structurally efficient and are restored to stiffness and strength values nearly equal to or greater than those of the control beams. It was found that the efficiency of the strengthening technique by CFRP in flexure varied depending on the length. The main failure mode in the experimental work was plate debonding in retrofitted beams.

Panda et al. (2011) have investigated the performance of 2500mm long reinforced concrete (RC)T-beams strengthened in shear using epoxy bonded glass fiber fabric. The experimental program consisted of testingof 18 full scale simply supported RC T-beams. The experimental result indicates that RC T-beams strengthened in shear with side-bonded GFRP sheet increases the effectiveness by 12.5% to 50%.

Deifalla et al. (2012) investigated on several cases of loading and geometrical configurations, flexure beams, and girders are subjected to combined shear and torsion.. Four strengthening techniques using carbon FRPs were tested. The experimental results were reported and analyzed to assess the effectiveness of the proposed strengthening techniques. An innovative strengthening technique namely the extended U-jacket showed promising results in terms of strength and ductility while being quite feasible for strengthening.

2.4 Strengthening of RC Rectangular and T- Beams with web opening:

Generally, in the construction of modern buildings, a network of pipes and ducts are necessary to accommodate essential services like water supply, sewage, ventilating, air-conditioning, electricity, telephone, and computer network. Openings in concrete beams enable the installation of these services.

Shanmugamt et al. (1988) have studied the strength of fiber reinforced concrete deep beams with openings. In this study, nine beams were tested to failure, all the beams were of the same dimensions having a length of 1550 mm, overall depth of 650 mm and width of 80 mm. Steel fiber content in all the beams was kept the same equal to 1% by volume. Two rectangular openings, one in each shear span, were placed symmetrically about the vertical axis in each of the beams. The beams were simply supported on a clear span of 1300 mm, and are tested under two point loading. The experimental results presented here confirm previous findings, i.e., the effect of opening on the behaviour and ultimate shear strength of deep beams depends primarily on the extent to which it intercepts the natural load path and the location at which this interception occurs.

Mansur (1998) has studied effect of openings on the behaviour and strength of reinforced concrete (RC) beams in shear. In this study, the behaviour and design of a beam containing a transverse opening and subjected to a predominant shear are briefly reviewed. Based on the observed structural response of the beam, suitable guidelines are proposed for classifying an opening as small or large. For small openings, a design method compatible with the current design philosophy for shear is proposed and illustrated by a numerical design example. In the method proposed, the maximum shear allowed in the section to avoid diagonal compression failure has been assumed to be the same as that for solid beam except for considering the net section through the opening.

Mansur (2006) has studied the design of reinforced concrete beams with web openings. To investigate the problem of openings in beams, the author initiated a research program in the early 1980s. Since then extensive research has been carried out giving a comprehensive coverage on both circular and large rectangular openings under various combinations of bending, shear and torsion. In this study, major findings relevant to the analysis and design of such beams under the most commonly encountered loading case of bending and shear are extracted and summarized. An attempt has been made to answer the frequently asked questions related to creating an opening in an already constructed beam and how to deal with multiple openings. It has been shown that the design method for beams with large openings can be further simplified without sacrificing rationality and having unreasonable additional cost.

Maaddawy et al. (2009) have studied the results of a research work aimed at examining the potential use of upgrade reinforced concrete (RC) deep beams with openings. A total of 13 deep beams with openings were constructed and tested under four-point bending. Test specimen had a cross-section of 80 x 500 mm and a total length of 1200mm. Two square openings, one in each shear span, were placed symmetrically about the mid-point of the beam. Test parameters included the opening size, location, and the presence of the CFRP sheets. The strength gain caused by the CFRP sheets was in the range of 35 - 73%. Based on the test results concluded that, the CFRP shear-strengthened RC deep beams with openings failed suddenly due to a formation of diagonal shear cracks in the top and bottom chords of the opening. In all strengthened beams, the concrete was pulled out from the U-shaped CFRP jacket wrapped around the top chord of the opening. The shear strength gain caused by CFRP sheets was in the range of 66 - 71% when the opening was located at the mid-point of the shear span. The shear strength gain was maximum (72%) when the opening was located at the top of the beam where most of the shear force was carried by the bottom chord that was fully wrapped with CFRP. Only a strength gain of 35% was recorded for the beam with bottom openings because most of the shear force was carried by the top chord that had a U-shaped CFRP sheet.

2.5 Critical Observations

The following critical observations are made from the review of existing literature in the area of reinforced concrete (RC) beams strengthened with epoxy-bonded FRP.

- Most of the research efforts have been made to study the flexural and shear behaviour of RC rectangular beams strengthened with fiber reinforced polymer (FRP) composites.
- Despite the growing number of field applications, there is limited number of reports on shear behaviour of strengthened RC T-beams using externally bonded FRP composites.
- A limited works have been reported on strengthening of RC T-beams with web openings.
- There is a gain in shear capacity of RC beams when strengthened with FRP composites, peeling of FRP sheets from main concrete has been reported due to improper anchorage.
- The study on anchorage system used for the prevention of debonding of FRP and concrete on shear behaviour of RC beams is limited.
- Many researchers are of the opinion that the previous design provisions do not have comprehensive understanding of the shear behaviour.

2.6 Scope of the present Investigation

Based on the critical review of the existing literature and to fulfil the objective outlined earlier, the scope of the present work is defined as follows:

- To study the behaviour of shear deficient RC T-beams with transverse openings in web portion.

- To study the contribution of GFRP composites on ultimate load carrying capacity and failure pattern of reinforced concrete beams.

- To study the effect of anchorage system used for prevention of debonding of FRP and concrete on the shear capacity of RC T-beams.
- To know the behaviour of reinforced concrete T- beams, retrofitted with GFRP.
- To know the practical feasibility of FRP in the construction industry.

CHAPTER – 3

EXPERIMENTAL PROGRAM

CHAPTER - 3

EXPERIMENTAL PROGRAM

The objective of the experimental program is to study the effect of externally bonded (EB) fiber reinforced polymer (FRP) sheets on the shear capacity of reinforced concrete T-beam with a transverse opening in shear span under static loading condition. Eleven number of reinforced concrete T-beams are cast and tested up to failure by applying symmetrical four-point static loading system. These beams were divided into 2 groups designated as A and B. The difference between two groups was in transverse steel reinforcement. Out of eleven number of beams, four beams were not strengthened by FRP and in that two beams were considered as a control beams and two beams were solid beams without transverse opening, whereas all other seven beams were strengthened with externally bonded GFRP sheets in shear zone of the beam.

The variables investigated in this research study included steel stirrups (i.e., beams with and without steel stirrups), shear span-to depth ratio (i.e., a/d ratio 2.66 versus 2), and end anchor (i.e., U-wrap with and without end anchor).

3.1TEST SPECIMENS

All eleven reinforced concrete T-beams had a span of 1300 mm, 150mm wide web, 350mm wide flange, 125mm deep web, 50mm deep flange and effective depth of 125mm.

The arrangement of reinforcement of beams under group-A consists of 2numbers of 20mm φ and 1number of 10mm φ HYSD bars as tension reinforcement, four bars of 8mm φ are also provided as hang up bars and without any shear reinforcement.

The arrangement of reinforcement of beams under group-B consists of 2numbers of 20mm φ and 1number of 10mm φ HYSD bars as tension reinforcement, four bars of 8mm φ are also provided as hang up bars and 8mm φ bars are provided as shear reinforcement at 200 mm spacing.

3.2 MATERIAL PROPERTIES

3.2.1 Concrete

For conducting experiment, the proportions in the concrete mix are tabulated in Table 3.1 as per IS: 456-2000. The water cement ratio is fixed at 0.55. The mixing is done by using concrete mixture. The beams are cured for 28 days. For each beam six 150x150x150 mm concrete cube specimens and six 150x300 mm cylinder specimens were made at the time of casting and were kept for curing, to determine the compressive strength of concrete at the age of 7 days & 28 days are shown in table 3.2.

Table 3.1 Nominal Mix Proportions of Concrete

Description	Cement	Sand (Fine Aggregate)	Coarse Aggregate	Water
Mix Proportion (by weight)	1	1.67	3.33	0.6
Quantities of materials for one specimen beam (kg)	44.4	74.11	147.85	22.5

The compression tests on control and strengthened specimen of cubes are performed at 7 days and 28 days. The test results of cubes are presented in Table 3.2.

Table 3.2 Test Result of Cubes after 28 days

Specimen Name		Size of Cube Specimen	Size of Cylinder Specimen	Average Cube Compressive Strength (MPa)	Average Cylinder Compressive Strength (MPa)
Group-A	Solid beam	150x150x150	150φ x300	35.23	25.15
	CBA	150x150x150	150φ x300	35.88	20.93
	SBA2-1	150x150x150	150φ x300	36.5	21.86
	SBA2-2	150x150x150	150φ x300	34.87	23.72
	SBA2-3	150x150x150	150φ x300	36.47	20.46
	SBA4-1	150x150x150	150φ x300	37.86	26.82
Group-B	Solid beam	150x150x150	150φ x300	30.83	21.08
	CBB	150x150x150	150φ x300	32.56	22.75
	SBB2-1	150x150x150	150φ x300	31.89	20.74
	SBB2-2	150x150x150	150φ x300	35.4	23.5
	SBB2-3	150x150x150	150φ x300	36.77	24.87

3.2.2 Cement

Cement is a material, generally in powered form, which can be made into a paste usually by the addition of water and, when molded or poured, will set into a solid mass. Numerous organic compounds used for an adhering, or fastening materials, are called cements, but these are classified as adhesives, and the term cement alone means a construction material. The most widely used of the construction cements is Portland cement. It is bluish-gray powered obtained by finely grinding the clinker made by strongly heating an intimate mixture of calcareous and argillaceous minerals. Portland Slag Cement (PSC) Konark Brand was used for this investigation. It is having a specific gravity of 2.96.

3.2.3 Fine Aggregate

Fine aggregate/sand is an accumulation of grains of mineral matter derived from disintegration of rocks. It is distinguished from gravel only by the size of the grains or particles, but is distinct from clays which contain organic material. Sand is used for making mortar and concrete and for polishing and sandblasting. Sands containing a little clay are used for making molds in foundries. Clear sands are employed for filtering water. Here, the fine aggregate/sand is passing through 4.75 mm sieve and having a specific gravity of 2.64. The grading zone of fine aggregate is zone III as per Indian Standard specifications IS: 383-1970.

3.2.4 Coarse Aggregate

Coarse aggregates are the crushed stone is used for making concrete. The commercial stone is quarried, crushed, and graded. Much of the crushed stone used is granite, limestone, and trap rock. The coarse aggregates of two grades are used one retained on 10 mm size sieve and another grade contained aggregates retained on 20 mm size sieve. The maximum size of coarse aggregate was 20 mm and is having specific gravity of 2.88 grading confirming to IS: 383-1970.

3.2.5 Water

Water fit for drinking is generally considered good for making the concrete. Water should be free from acids, alkalis, oils, vegetables or other organic impurities. Soft water produces weaker concrete. Water has two functions in a concrete mix. Firstly, it reacts chemically with the cement to form a cement paste in which the inert aggregates are held in suspension until the cement paste has hardened. Secondly, it serves as a vehicle or lubricant in the mixture of fine aggregates and cement. Ordinary clean portable tap water is used for concrete mixing in all the mix.

3.2.6 Reinforcing Steel

High-Yield Strength Deformed (HYSD) bars confirming to IS 1786:1985. The longitudinal steel reinforcing bars were deformed, high-yield strength, with 20 mm and 10 mm diameter. The stirrups were made from deformed steel bars with 8 mm diameter.

Three coupons of steel bars were tested and yield strength of steel reinforcements used in this experimental program is determined under uniaxial tension an accordance with ASTM specifications. The proof stress or yield strength of the specimens are averaged and shown in Table 3.5. The modulus of elasticity of steel bars was 2×10^{5} MPa.

Table 3.3 Tensile Strength of reinforcing steel bars

Sl. no. of sample	Diameter of bar (mm)	0.2% Proof stress (N/mm^2)	Avg. Proof Stress (N/mm^2)
1	20	475	470
2	20	472	
3	20	463	
4	10	530	529
5	10	535	
6	10	521	
7	8	520	523
8	8	527	
9	8	521	

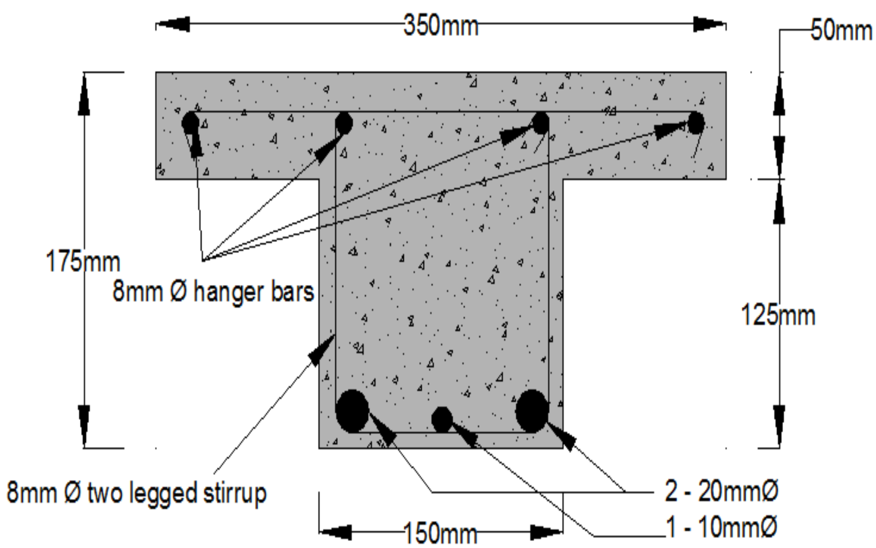


Figure 3-1. Detailing of Reinforcement

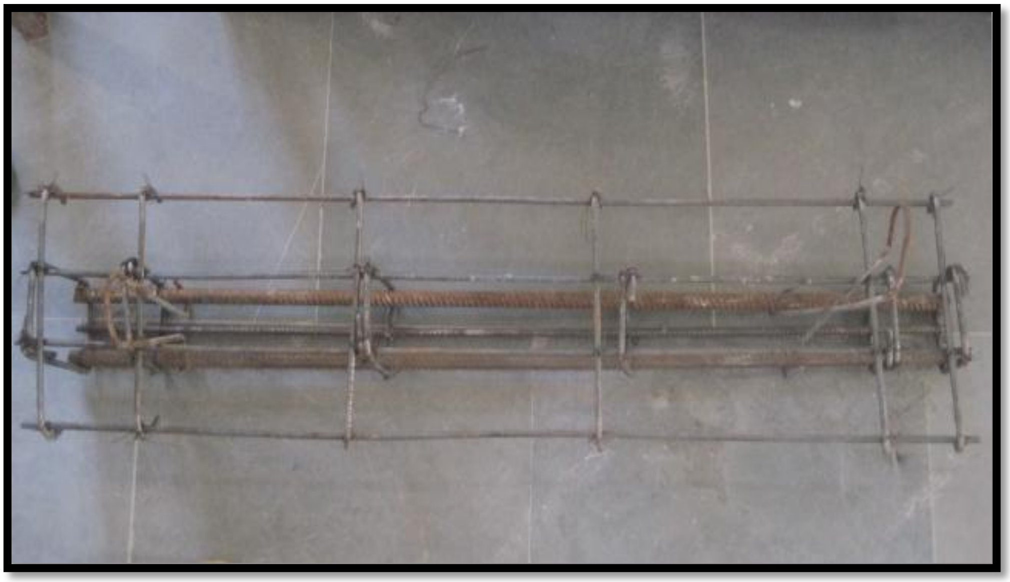


Figure 3-2. Reinforcement Detailing of T- Beam

3.2.7 Fiber Reinforced Polymer (FRP)

Continuous fiber reinforced materials with polymeric matrix (FRP) can be considered as composite, heterogeneous, and anisotropic materials with a prevalent linear elastic behavior up to failure. Normally, Glass and Carbon fibers are used as reinforcing material for FRP. Epoxy is used as the binding material between fiber layers.

For this study, one type of FRP sheet was used during the tests i.e., a bidirectional FRP with the fiber oriented in both longitudinal and transverse directions, due to the flexible nature and ease of handling and application, the FRP sheets are used for shear strengthening. Throughout this study, E-glass was used manufactured by Owens Corning.

3.2.8 Epoxy Resin

The success of the strengthening technique primarily depends on the performance of the epoxy resin used for bonding of FRP to concrete surface. Numerous types of epoxy resins with a wide range of mechanical properties are commercially available in the market. These epoxy resins are generally available in two parts, a resin and a hardener. The resin and hardener used in this study are Araldite LY 556 and hardener HY 951 respectively in a proportion of 10:1.

3.2.9 Casting of GFRP Plate for tensile strength

There are two basic processes for moulding, that is, hand lay-up and spray-up. The hand lay-up process is the oldest, simplest, and most labour intense fabrication method. This process is the most common in FRP marine construction. In hand lay-up method liquid resin is placed along with reinforcement (woven glass fiber) against finished surface of an open mould. Chemical reactions in the resin harden the material to a strong, light weight product. The resin serves as the matrix for the reinforcing glass fibers, much as concrete acts as the matrix for steel reinforcing rods. The percentage of fiber and matrix was 50:50 by weight.

The following constituent materials are used for fabricating the GFRP plate:

i. Glass FRP (GFRP)

ii. Epoxy as resin

iii. Hardener as diamine (catalyst)

iv. Polyvinyl alcohol as a releasing agent

Contact moulding in an open mould by hand lay-up was used to combine plies of woven roving in the prescribed sequence. A flat plywood rigid platform was selected. A plastic sheet was kept on the plywood platform and a thin film of polyvinyl alcohol was applied as a releasing agent by use of spray gun. Laminating starts with the application of a gel coat (epoxy and hardener) deposited on the mould by brush, whose main purpose was to provide a smooth external surface and to protect the fibers from direct exposure to the environment. Ply was cut from roll of woven roving. Layers of reinforcement were placed on the mould at top of the gel coat and gel coat was applied again by brush. Any air bubble which may be entrapped was removed using serrated steel rollers. The process of hand lay-up was the continuation of the above process before the gel coat had fully hardened. Again, a plastic sheet was covered the top of the plate by applying polyvinyl alcohol inside the sheet as releasing agent. Then, a heavy flat metal rigid platform was kept top of the plate for compressing purpose. The plates were left for a minimum of 48 hours before being transported and cut to exact shape for testing.

Plates of 1 layer, 2 layers, 4 layers, 6 layers and 8 layers were casted and three specimens from each thickness were tested.

Figure 3-3. Specimens for tensile testing of woven Glass/Epoxy composite

Figure 3-4. Experimental setup of INSTRON universal testing Machine of 600 kN capacity

Figure 3-5. Specimen during testing

3.2.10 Determination of Ultimate Stress, Ultimate Load & Young's Modulus of FRP

The ultimate stress, ultimate load and young's modulus was determined experimentally by performing unidirectional tensile tests on specimens cut in longitudinal and transverse directions. The specimens were cut from the plates by diamond cutter or by hex saw. After cutting by hex saw, it was polished with the help of polishing machine. At least three replicate sample specimens were tested and mean values adopted. The dimensions of the specimens are shown in below table 3.4.

Table 3.4 Size of the Specimens for tensile test

No. of Layers	Length of sample (mm)	Width of sample (mm)	Thickness of sample (mm)
1	250	25	0.7
2	250	25	1
4	250	25	1.7
6	250	25	2.1
8	250	25	3.1

For measuring the tensile strength and young's modulus, the specimen is loaded in INSTRON 600 kN in Production Engineering Lab, NIT, Rourkela. Specimens were gripped in the fixed upper jaw first and then gripped in the movable lower jaw. Gripping of the specimen should be proper to prevent the slippage. Here, it is taken as 50 mm from the each side. Initially, the strain is kept zero. The load, as well as the extension, was recorded digitally with the help of a load cell and an extensometer respectively. From these data, stress versus strain graph was plotted, the initial slope of which gives the young's modulus. The ultimate stress and ultimate load were obtained at the failure of the specimen. The average value of each layer of the specimens is given in the below Table 3.5.

Table 3.5 Result of the Specimens

GFRP plate of	Ultimate Stress (MPa)	Ultimate Load (N)	Young's Modulus (MPa)
1 layer	137.9	2760	5658
2 layers	167.7	4190	10020
4 layers	210.1	9400	9493
6 layers	276.8	13840	11000
8 layers	228.7	17720	9253

3.2.11 Form Work

Fresh concrete being plastic in nature requires good form work to mould it to the required shape and size. So the form work should be rigid and strong to hold the weight of wet concrete without bulging anywhere. The joints of the form work are sealed to avoid leakage of cement slurry. Mobil oil was then applied to the inner faces of form work. The bottom rest over thick polythene sheet lead over the rigid floor. The reinforcement cage was then lowered, placed in position inside the side work carefully with a cover of 20mm on sides and bottom by placing concrete cover blocks.

Figure 3-6. Steel Frame Used For Casting of RC T-Beam

3.1.12 Mixing of Concrete

Mixing of concrete is done thoroughly with the help of standard concrete mixer machine, to ensure that a uniform quality of concrete is obtained. First coarse and fine aggregates are fed alternately, followed by cement. Then required quantity of water is slowly added into the mixer to make the concrete workable until a uniform colour is obtained. The mixing is done for two minutes after all ingredients are fed inside the mixer as per IS: 456-2000.

3.1.13 Compaction

All specimens were compacted by using 30mm size needle vibrator for good compaction of concrete, and sufficient care was taken to avoid displacement of the reinforcement cage inside the form work. Finally, the surface of concrete was leveled and smoothened by metal trowel and wooden float. After seven hours, the specimen detail and date of concreting was written on top surface to identify it properly.

3.1.14 Curing of Concrete

Curing is done to prevent the loss of water which is essential for the process of hydration and hence for hardening. Usually, curing starts as soon as the concrete is sufficiently hard. Here, curing is done by spraying water on the jute bags or by spending wet hessians cloth over the surface for a period of 28 days.

3.1.15 Strengthening of Beams with FRP sheets

All the loose particles of concrete surface at the bottom sides of the beam were chiseled out by using a chisel. Then the required region of concrete surface was made rough using a coarse sand paper texture and cleaned with an air blower to remove all dirt and debris particles. Once the surface was prepared to the required standard, the epoxy resin was mixed in accordance with manufacturer's instructions. The mixing is carried out in a plastic container (100 parts by weight of Araldite LY 556 to 10 parts by weight of Hardener HY 951) and was continued until the mixture was in uniform. After their uniform mixing, the fabrics are cut according to the size then the epoxy resin is applied to the concrete surface. After their uniform mixing, the fabrics are cut according to the size then the epoxy resin is applied to the concrete surface. Then the GFRP sheet is placed on top of epoxy resin coating and the resin is squeezed through the roving of the fabric with the roller .Air bubbles entrapped at the epoxy/concrete or epoxy/fabric interface are eliminated. Then the second layer of the epoxy resin was applied and GFRP sheet was then placed on top of epoxy resin coating and the resin was squeezed through the roving of the fabric with the roller and the above process was repeated. The composite laminate was attached starting at one end and applying enough pressure to press out any excess epoxy from the sides of the laminate. During hardening of the epoxy, a constant uniform pressure is applied on the composite fabric surface in order to extrude the excess epoxy resin and to ensure good contact between the epoxy, the concrete and the fabric. This operation is carried out at room temperature. Concrete beams strengthened with glass fiber fabric are cured for minimum of one week at room temperature before testing.

3.3 EXPERIMENTAL SETUP

All the specimens except SBA3 and SBB3 are tested as simple RC T-beams using four-point static loading frame with shear span to effective depth ratio (*a/d*) equal to 2.66 and SBA3 and SBB3 are having a/d ratio equal to 2. The tests were carried out at the 'Structural Engineering' Laboratory of Civil Engineering Department, NIT Rourkela. The testing procedure for the entire specimen is same. After the curing period of 28 days are over, then the beam surface is cleaned with the help of sand paper for clear visibility of cracks. Figure 3-10 shows the details of the test setup. A load cell with a capacity of 500 kN and attached to a hydraulic jack was used to measure the load during testing.

Four-point loading is conveniently provided by the arrangement shown in Figure 3-10. The load is transmitted through a load cell and spherical seating on to a spreader beam.

This spreader beam is installed on rollers seated on steel plates bedded on the test member with cement in order to provide a smooth leveled surface. The test member is supported on roller bearings acting on similar spreader plates. The loading frame must be capable of carrying the expected test loads without significant distortion. Ease of access to the middle third for crack observations, deflection readings and possibly strain measurements is an important consideration, as is safety when failure occurs. The specimen is placed over the two steel rollers bearing leaving 150mm from the ends of the beam. The remaining 1000mm is divided into three equal parts of 333mm as shown in the figure 3-10. Load is applied by hydraulic jack of capacity 500kN. Lines are marked on the beam to be tested at L/3, L/2, & 2L/3 locations from the left support (L=1300mm), three dial gauges are used for recording the deflection of the beams. One dial gauge is placed just below the centre of the beam, i.e. at L/2 distance and the remaining two dial gauges are placed just below the point loads, i.e. at L/3 and 2L/3 to measure the deflections.

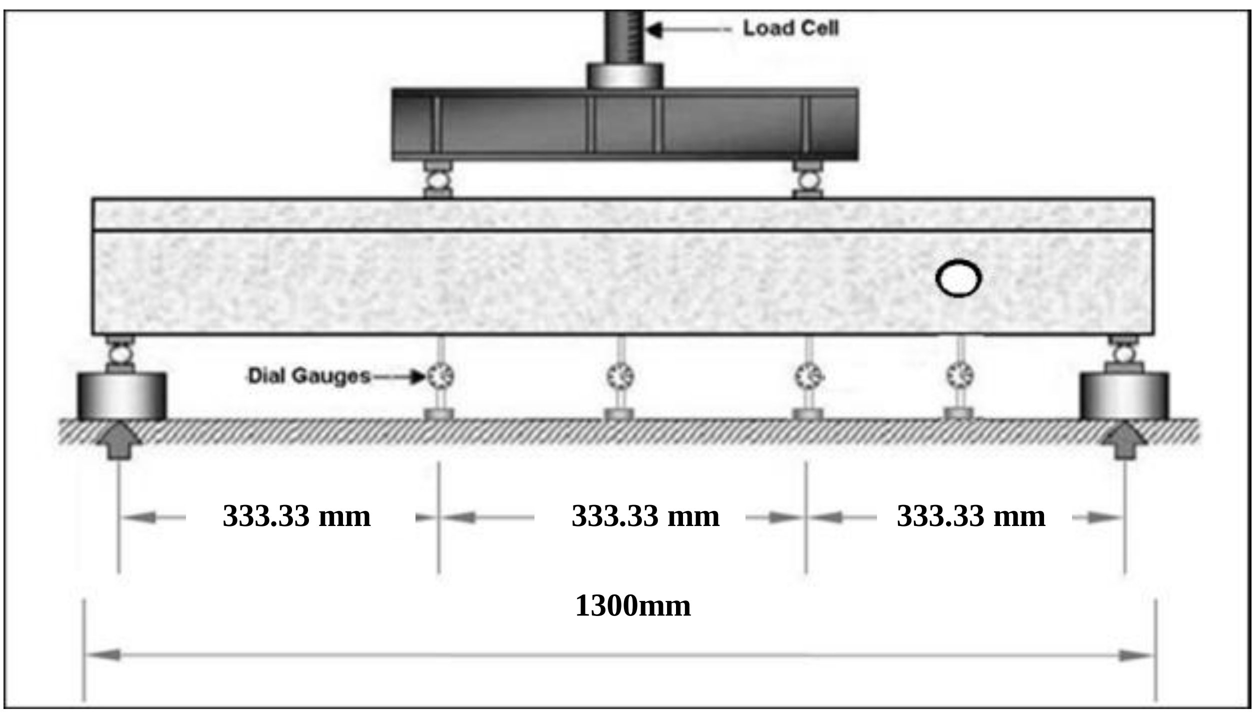


Figure 3-7. Details of the Test setup with location of dial gauges

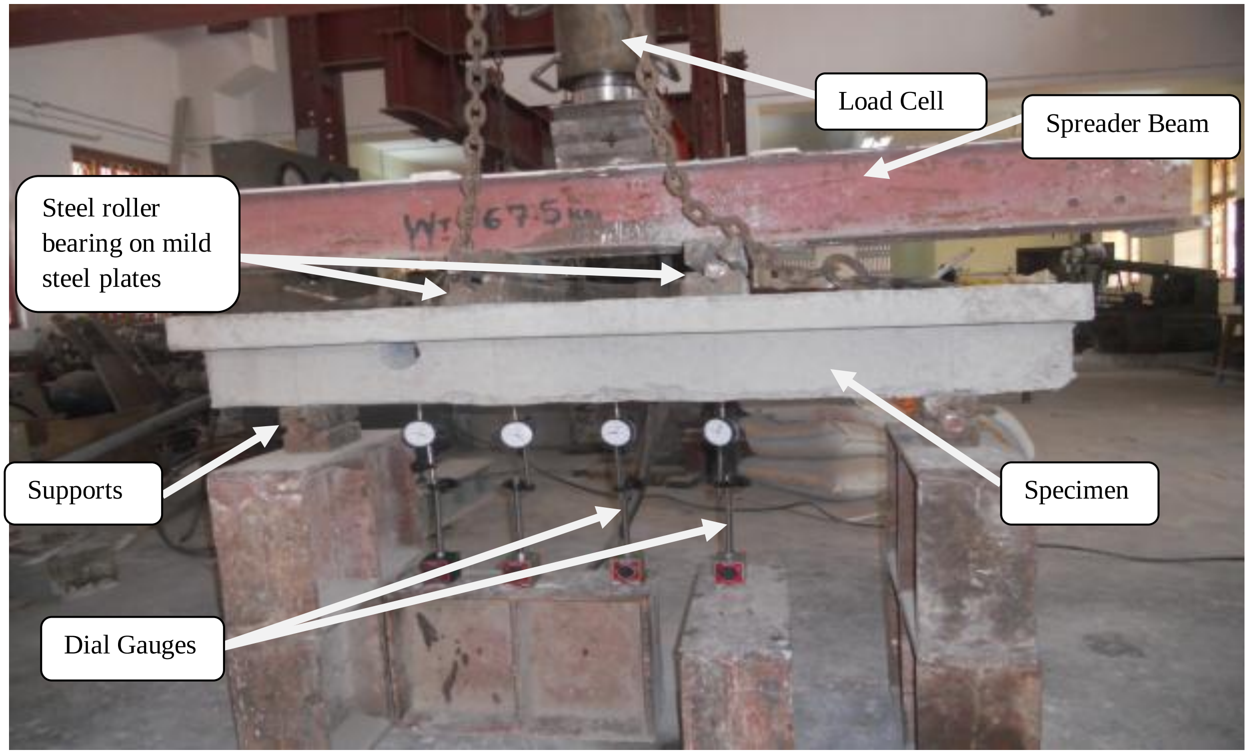


Figure 3-8. Experimental Setup for testing of beams

3.4 DESCRIPTION OF SPECIMENS

The experimental program consists of 11number of simply supported RC T-beams divided into two groups as mentioned earlier.

3.4.1 GROUP-A

This group having 6 number of beams with 2-20mmφ and 1-10mmφ as longitudinal reinforcement and without any shear reinforcement to make the beams deficient in shear.

3.4.1.1 SOLID BEAM

The solid beam not strengthened with GFRP. It is designed to know the behavior of the beam without opening under four-point static loading test. It is totally weak in shear mainly on shear span where the transverse opening is provided i.e., shear deficient beam shown in figure 3-9.

Figure 3-9. Model of T-beam without GFRP and transverse opening – Solid beamA

3.4.1.2 CONTROL BEAM (CBA)

The control beam (CB) not strengthened with GFRP. It is designed to achieve the shear failure under four-point static loading test. It is totally weak in shear mainly on shear span where the transverse opening is provided i.e., shear deficient beam shown in figure 3-10.

Figure 3-10. Model of T-beam without GFRP – CBA

3.4.1.3 STRENGTHENED BEAM 1 (SBA2-1)

The beam (SB1) is modeled with two layers of GFRP having U-wrap on bottom and web portions of the shear span on which the transverse opening is provided as show in figure 3-11. The same four-point static loading is applied at the middle-third locations.

Figure 3-11. Model of T-beam with GFRP – SBA2-1

3.4.1.4 STRENGTHENED BEAM 2 (SBA2-2)

The beam (SB2) was strengthened by applying two layers of GFRP on web portions on shear span region (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates of 8 layers instead of steel plates with bolts to prevent debonding and to increase the strength of the beam. Set-up is shown in figure 3-12.

Figure 3-12. Model of T-beam with GFRP with anchorage system – SBA2-2

3.4.1.5 STRENGTHENED BEAM 3 (SBA2-3)

The beam (SB3) was strengthened by applying two layers of GFRP on web portions on shear span region with flange anchorage system by using GFRP plates of 8 layers instead of steel plates with bolts to prevent debonding. Shear span is considered as 250mm from the supports, so that the a/d ratio get changed. Set-up as shown in figure 3-13.

Figure 3-13. Model of T-beam with GFRP with anchorage system – SBA2-3

3.4.1.6 STRENGTHENED BEAM 4 (SBA4-1)

The beam (SB4) was strengthened by applying four layers of GFRP on web portions on shear span region (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates of 8 layers instead of steel plates with bolts to prevent debonding as shown in figure 3-14.

Figure 3-14. Model of T-beam with GFRP with anchorage system – SBA4-1

3.4.2 GROUP-B

This group having 5 number of beams with 2-20mmφ and 1-10mmφ as longitudinal reinforcement and 8mmφ bars are provided at 200mm spacing as shear reinforcement to check the behavior of the beam by providing shear reinforcement.

3.4.2.1 SOLID BEAM

The solid beam not strengthened with GFRP. It is designed to know the behavior of the beam without opening under four-point static loading test. It is totally weak in shear mainly on shear span where the transverse opening is provided i.e., shear deficient beam shown in figure 3-15.

Figure 3-15. Model of T-beam without GFRP and transverse opening – Solid beamB

3.4.2.2 CONTROL BEAM (CBB)

The control beam (CB) not strengthened with GFRP. It is designed to achieve the shear failure under four-point static loading test. It is totally weak in shear mainly on shear span where the transverse opening is provided i.e., shear deficient beam shown in figure 3-16.

Figure 3-16. Model of T-beam without GFRP – CBB

3.4.2.3 STRENGTHENED BEAM 1 (SBB2-1)

The beam (SB1) is modeled with two layers of GFRP having U-wrap on bottom and web portions of the shear span on which the transverse opening is provided as show in figure 3-17. The same four-point static loading is applied at the middle-third locations.

Figure 3-17. Model of T-beam with GFRP – SBB2-1

3.4.2.4 STRENGTHENED BEAM 2 (SBB2-2)

The beam (SB2) was strengthened by applying two layers of GFRP on web portions on shear span region (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates with steel bolts to prevent debonding and to increase the strength of the beam. Set-up is shown in figure 3-18.

Figure 3-18. Model of T-beam with GFRP with anchorage system – SBB2-2

3.4.2.5 STRENGTHENED BEAM 3 (SBB2-3)

The beam (SB4) was strengthened by applying two layers of GFRP on web portions on shear span region with flange anchorage system by using GFRP plates with bolts to prevent debonding. Shear span is considered as 250mm from the supports, so that the a/d ratio gets changed. Set-up as shown in figure 3-19.

Figure 3-19. Model of T-beam with GFRP with anchorage system – SBB2-3

3.5 SUMMARY

Eleven beams were tested in this experimental investigation. One control beams was tested, seven beams were strengthened with different orientations of GFRP, three beams were strengthened with epoxy bonded GFRP with anchorage system to avoid debonding and other two beams were strengthened with GFRP which has a opening in the shear region and with an anchorage system. The detail descriptions of above mentioned beams are presented in Table 3.8.

Table 3.6 Beam test parameters and material properties

Beam ID		f_c (MPa)	Tension Reinforcement	Yield Stress, f_y (MPa)	Material Type	Sheet Thickness (mm)	Strengthening system with GFRP sheets
Group-A	**Solid beam**	25.15	2-20mm φ, 1-10mm φ	470, 529	-	-	Solid beam
	CBA	20.93	2-20mm φ, 1-10mm φ	470, 529	--	--	Control Beam
	SBA2-1	21.86	2-20mm φ, 1-10mm φ	470, 529	GFRP	1	Two layers bonded to the bottom and sides of shear span only opening side of beam (U-shape)
	SBA2-2	23.72	2-20mm φ, 1-10mm φ	470, 529	GFRP	1	Two layers continuous bonded to the bottom and sides of shear span of beam with flange anchorage system with GFRP plates

	SBA2-3	20.46	2-20mmφ, 1-10mm φ	470, 529	GFRP	1	Two layers continuous bonded to the bottom and sides of shear span of beam with flange anchorage system with GFRP plates for a shear span of 250mm
	SBA4-1	26.82	2-20mm φ, 1-10mm φ	470, 529	GFRP	1.7	Four layers continuous bonded to the bottom and sides of shear span of beam with flange anchorage system with GFRP plates
Group-B	**Solid beam**	21.08	2-20mm φ, 1-10mm φ	470, 529	-	-	Solid beam
	CBB	22.75	2-20mm φ, 1-10mm φ	470, 529	-	-	Control Beam
	SBB2-1	20.74	2-20mm φ, 1-10mm φ	470, 529	GFRP	1	Two layers bonded to the bottom and sides of shear span only opening side of beam (U-shape)
	SBB2-2	23.5	2-20mm φ, 1-10mm φ	470, 529	GFRP	1	Two layers continuous bonded to the bottom and sides of shear span of beam with flange anchorage system with GFRP plates
	SBB2-3	24.87	2-20mm φ, 1-10mm φ	470, 529	GFRP	1	Two layers continuous bonded to the bottom and sides of shear span of beam with flange anchorage system with GFRP plates for a shear span of 250mm

CHAPTER-4

TEST RESULTS&

DISCUSSIONS

CHAPTER - 4

TEST RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

In this chapter, the results obtained from the testing of eleven number RC T-Beams for the experimental program are interpreted. Their behaviours throughout the test are described with respect to initial crack load and ultimate load carrying capacity, deflection, crack pattern and modes of failure.

All the beams except the control beams (CB) and solid beams are strengthened with various patterns of GFRP sheets. All the beams except SBA2-3 and SBB2-3 were in both the groups having shear span of 333.33 mm and SB3 having shear span of 250 mm. Specimens of group-A were cast without stirrups and group-B casted with stirrups at 200mm spacing. In both the groups the beam designated as SBA2-1 and SBB2-1 were strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear of the beam where the transverse hole is provided. The beam SBA2-2 and SBB2-2 were strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear span (0 to L/3 and L/3 to 2L/3) of the beam with flange anchorage system. The beam SBA2-3 and SBB2-3 were strengthened with two layers of bi-directional GFRP sheets in the form of U-wrap on shear span for a width of 250 mm with flange anchorage system. SBA4-1 was strengthened with four layers of bi-directional GFRP sheets having U-wrap on shear span (0 to L/3 and L/3 to 2L/3) of the beam with flange anchorage system by using GFRP plates instead of steel plates.

4.2 Crack Behaviour and Failure Modes

The crack behaviour and failure modes of the eleven number of beams tested in the experimental program are described below.

4.2.1. GROUP-A

4.2.1.1Solid Beam

The solid beam was cast with same reinforcement used for control beam but no transverse hole is provided. Figure 4-1 (a) shows the experimental test setup of solid beam under four point loading. The first hair crack was visible in the shear span at a load of 110 kN as shown in figure 4-1(b). With further increase in load, the beam finally failed in shear at a load of 208 kN exhibiting a wider diagonal shear crack as shown in figure 4-1 (c). The first shear crack became the critical crack for the ultimate failure of the solid beam. There is a 17.3% increase in shear capacity over the control beam.

4-1 (a) Experimental Setup of beam Solid beam

4-1 (b) Hair line crack started at 110kN

4-1 (c) Widened crack at ultimate load

4.2.1.2 Control Beam (CBA)

The control beam (CB) was cast with a reinforcement of two numbers of 20 mm bar and one number of 10 mm bar on tension face. The stirrups were not provided in the beam to make it shear deficient. The beam was tested by applying the point loads gradually. Figure 4-2 (a) shows the experimental test setup of control beam under four point loading. The first hair crack was visible in the shear span at a load of 90 kN as shown in figure 4-2 (b). This crack appeared at the mid-height zone of the web of the beam. As the load increased beyond the first crack load, many inclined cracks were also developed and the first visible crack started widening and propagated With further increase in load, the beam finally failed in shear at a load of 172 kN exhibiting a wider diagonal shear crack as shown in figure 4-2 (c). The first shear crack became the critical crack for the ultimate failure of the control beam.

4-2(a) Experimental Setup of the CBA under four-point loading

4-2(b) Hair line crack started at 90kN in shear region   4-2(c) Crack Pattern near hole

4.2.1.3 Strengthened Beam 1 (SBA2-1)

The beam SB1 was strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear span in the portion where hole is provided. The first hair crack was visible inside the hole at a load of 90 kN as shown in figure4-3(a). The failure was initiated on the other side where strengthening is not provided as shown in figure 4-3(b). The failure pattern of SBA2-1 as shown in figure 4-3(c). SB1 finally failed at a load of 170kN. The strengthening of beam SBA2-1 with GFRP U-wraps resulted in a 4.7% increase in shear capacity over the control beam.

4-3(a) Hair line crack inside the hole 4-3(b) Shear failure

4-3(c) failure pattern of SBA2-1

4.2.1.4 Strengthened Beam 2 (SBA2-2)

The beam SB2 was strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear span (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates. The test setup of the beam is shown in figure 4-4 (a). The initial crack in concrete as appeared in SBA2-2 could not be observed because the shear zones were fully wrapped with GFRP sheets. But the failure mode was initiated inside of hole and it propagated outside by the debonding of GFRP sheets with concrete cover of beam SBA2-2 as shown in figure 4-4 (b). The debonding failure of GFRP sheet started at 130kN was followed by rupture of GFRP and the beam finally failed at load of 220 kN as shown in figure 4-4 (c). There is a 21.81% increase in shear capacity over the control beam.

4-4 (a) Experimental Setup of beam SBA2-2

4.4(b) debonding of GFRP sheet 4-4 (c) dedonding and rupture of GFRP sheet

4.2.1.5 Strengthened Beam 3 (SBA2-3)

The beam SB3 was strengthened with two layers of bi-directional GFRP sheets in the form of U-wrap with flange anchorage system on shear span for a distance of 250mm from the supports. The test setup of the beam is shown in figure 4-5 (a). The initial diagonal shear crack started at a load of 120kN. Debonding started at 170 kN as shown in figure 4-5(b) The load carrying capacity of beam SBA2-3 with GFRP strips is relatively close to that of beam SBA2-2. Failure of beam SBA2-3 occurred at an ultimate load of 210 kN by rupture as shown in figure 4-5 (c). The strengthening of beam SBA2-3 with GFRP U-wrap sheets with change in shear span on the beam resulted 18.09% increase in the shear capacity over the control beam.

4-5 (a) Experimental Setup of beam SBA2-3

4-5 (b) debonding of GFRP sheet

4-5 (c) rupture of GFRP sheet

4.2.1.6 Strengthened Beam 4 (SBA4-1)

The beam SB4 was strengthened with four layers of bi-directional GFRP sheets having U-wrap on shear span (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates. The test setup of the beam is shown in figure 4-6 (a). The initial crack in concrete started at a load of 140kN. But the failure mode was initiated inside of hole and it propagated outside by the debonding of GFRP sheets with concrete cover of beam SBA4-1 as shown in figure 4-6 (b). The concrete has been crushed as shown in figure4-6(c). The debonding failure of GFRP sheet started at 190kN was followed by rupture of GFRP and the beam finally failed at load of 230 kN .There is a 25.21% increase in shear capacity over the control beam.

4-6 (a) Experimental Setup of beam SBA4-1

4-6 (b) debonding of GFRP sheet

4-6 (c) crushing of concrete inside the hole

4.2.2 GROUP-B

4.2.2.1 Solid Beam

The solid beam was cast with same reinforcement used for control beam but no transverse hole is provided. Figure 4-7 (a) shows the experimental test setup of solid beam under four point loading. The first hair crack was visible in the shear span at a load of 110 kN as shown in figure 4-7 (b). With further increase in load, the beam finally failed in shear at a load of 240 kN exhibiting a wider diagonal shear crack as shown in figure 4-7 (c). The first shear crack became the critical crack for the ultimate failure of the solid beam. There is a 41.67% increase in shear capacity over the control beam.

4-7 (a) Experimental Setup of beam Solid beam B

4-7 (b) Hair line crack started at 110kN 4-7(c) Widened crack at ultimate load

4.2.2.2 Control Beam (CBB)

The control beam (CBB) was cast with a reinforcement of two numbers of 20 mm bar and one number of 10 mm bar on tension face. The stirrups were not provided in the beam to make it shear deficient. The beam was tested by applying the point loads gradually. Figure 4-8 (a) shows the experimental test setup of control beam under four point loading. The first hair crack was visible in the shear span at a load of 100 kN as shown in figure 4-8 (b). With further increase in load, the beam finally failed in shear at a load of 140 kN exhibiting a wider diagonal shear crack as shown in figure 4-8 (c).

4-8 (a) Experimental Setup of beam CBB

4-8 (b) Hair line crack started at 100kN

4-8 (c) Widened crack at ultimate load

4.2.2.3 Strengthened Beam 1 (SBB2-1)

The beam SB1 was strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear span in the portion where hole is provided. The first hair crack was visible inside the hole at a load of 110 kN. The failure was initiated on the other side where strengthening is not provided as shown in figure 4-9(a). The failure mode was initiated inside of hole and it propagated outside by the debonding of GFRP sheets with concrete cover as shown in figure 4-9(b). SBB2-1 finally failed at a load of 198kN due to rupture of GFRP .The strengthening of beam SBB2-1 with GFRP U-wraps resulted in a 29.29% increase in shear capacity over the control beam.

4.9 (a) Hair line crack started at 110kN   4-9 (b) debonding and rupture at ultimate load

4.2.2.4 Strengthened Beam 2 (SBB2-2)

The beam SB2 was strengthened with two layers of bi-directional GFRP sheets having U-wrap on shear span (0 to L/3 and 2L/3 to L distance from left support) with flange anchorage system by using GFRP plates. The test setup of the beam is shown in figure 4-10 (a). The initial crack in concrete as appeared in SBB2-2 could not be observed because the shear zones were fully wrapped with GFRP sheets. But the failure mode was initiated inside of hole and it propagated outside by the debonding of GFRP sheets with concrete cover of beam SBB2-2 as shown in figure 4-10(b). The debonding failure of GFRP sheet started at 130kN was followed by rupture of GFRP and the beam finally failed at load of 184 kN as shown in figure 4-10 (c). There is a 23.91% increase in shear capacity over the control beam.

4-10 (a) Experimental Setup of beam SBB2-2

4-10 (b) debonding of GFRP sheet

4-10 (c) rupture of GFRP sheet

4.2.2.5 Strengthened Beam 3 (SBB2-3)

The beam SB3 was strengthened with two layers of bi-directional GFRP sheets in the form of U-wrap with flange anchorage system on shear span for a distance of 250mm from the supports. The test setup of the beam is shown in figure 4-11 (a). The initial diagonal shear crack started at a load of 100kN. The load carrying capacity of beam SBB2-3 with GFRP strips is relatively close to that of beam SBB2-3. Failure started by debonding as shown in figure 4-11(b). Failure of beam SBB2-3 occurred at an ultimate load of 214 kN by rupture as shown in figure 4-11 (c). The strengthening of beam SBB2-3 with GFRP U-wrap sheets with change in shear span on the beam resulted in a 34.58% increase in the shear capacity over the control beam.

4-11 (a) Experimental Setup of beam SBB2-3

4.11(b) debonding of GFRP sheet

4-11 (c) rupture of GFRP sheet

4.3 Load-deflection history

The mid-span deflection of the control and strengthened beams were measured at different load steps and the deflections under the point loads and under centre of point load were also recorded. The load-deflection histories are illustrated in figures 4.12 to 4.22. It was observed that the deflection under the point load was less than that at the centre. These figures below show that the deflection curve was initially straight showing the linear relationship between the load and deflection and became non-linear with further increase in load.

4.3.1. Group-A:

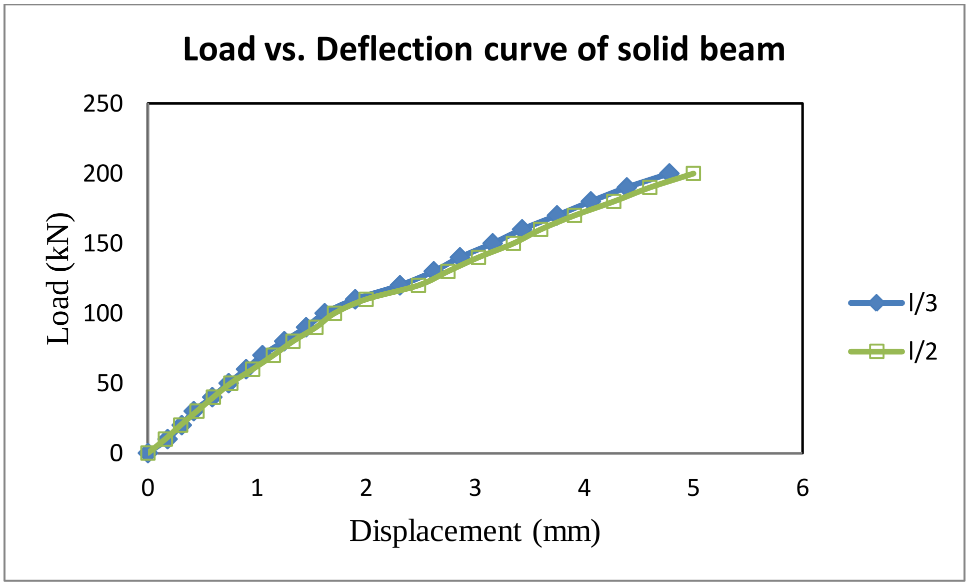


Figure 4-12. Load vs. Deflection Curve for Solid beam A

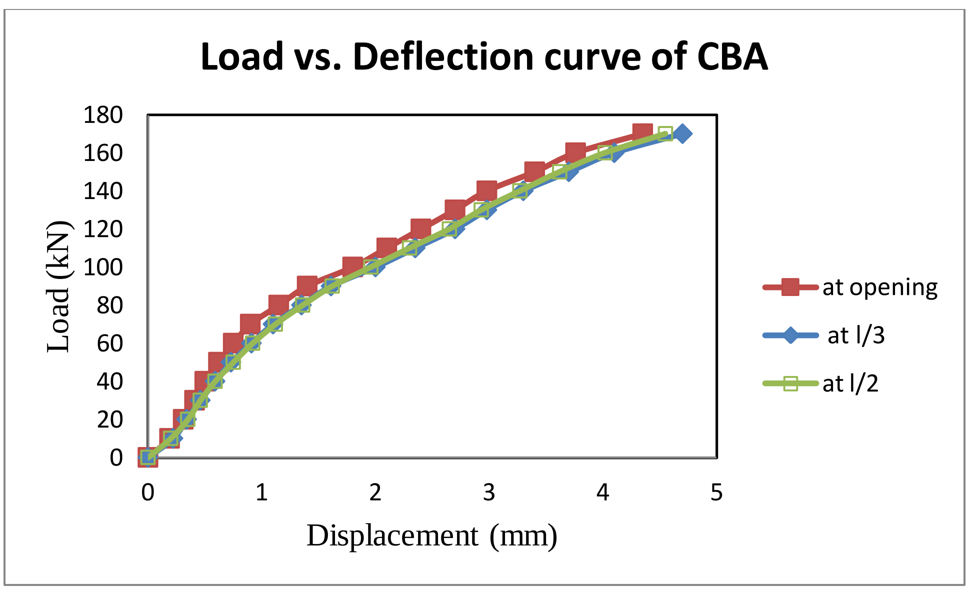


Figure 4-13. Load vs. Deflection Curve for CBA

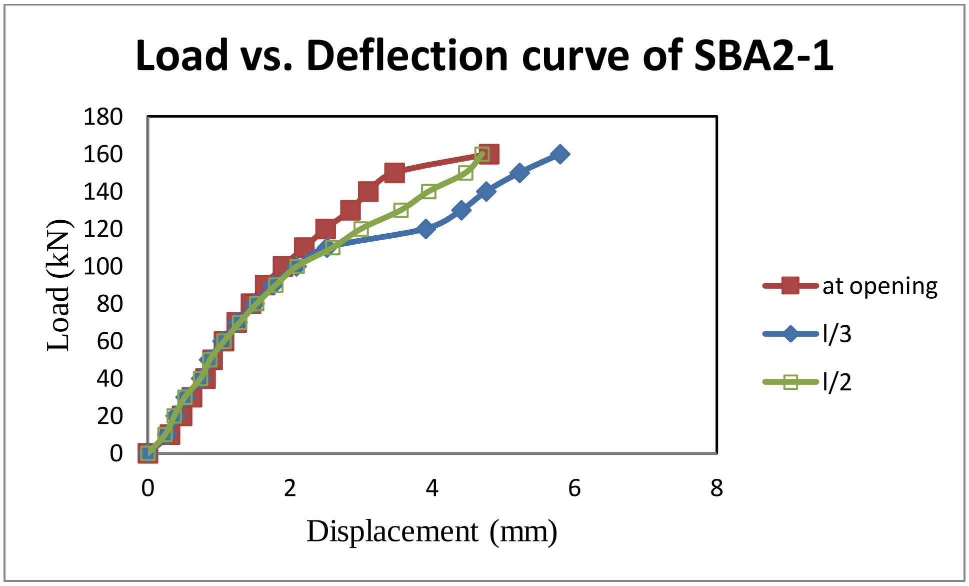


Figure 4-14. Load vs. Deflection Curve for SBA2-1

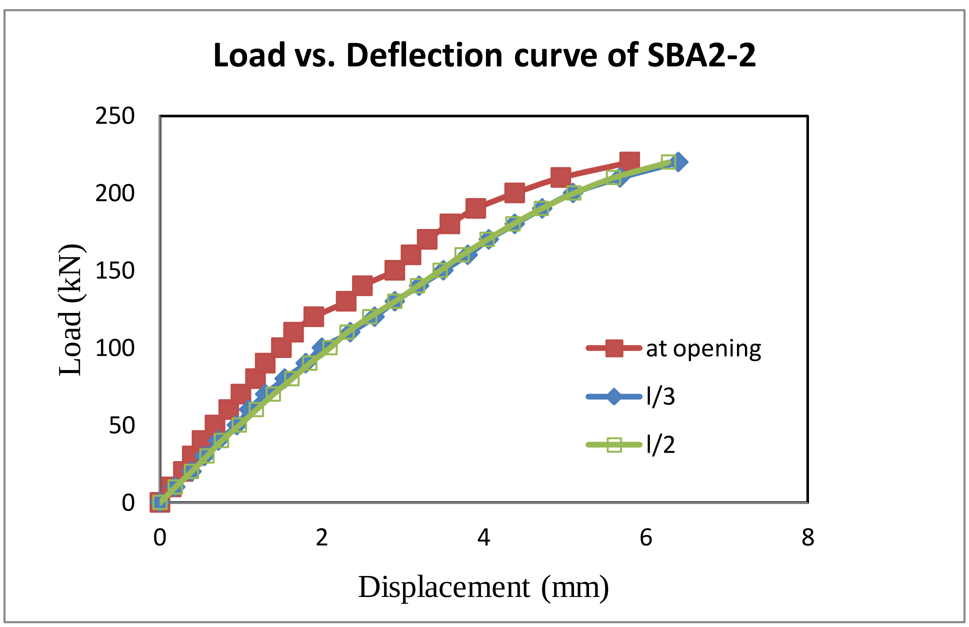


Figure 4-15. Load vs. Deflection Curve for SBA2-2

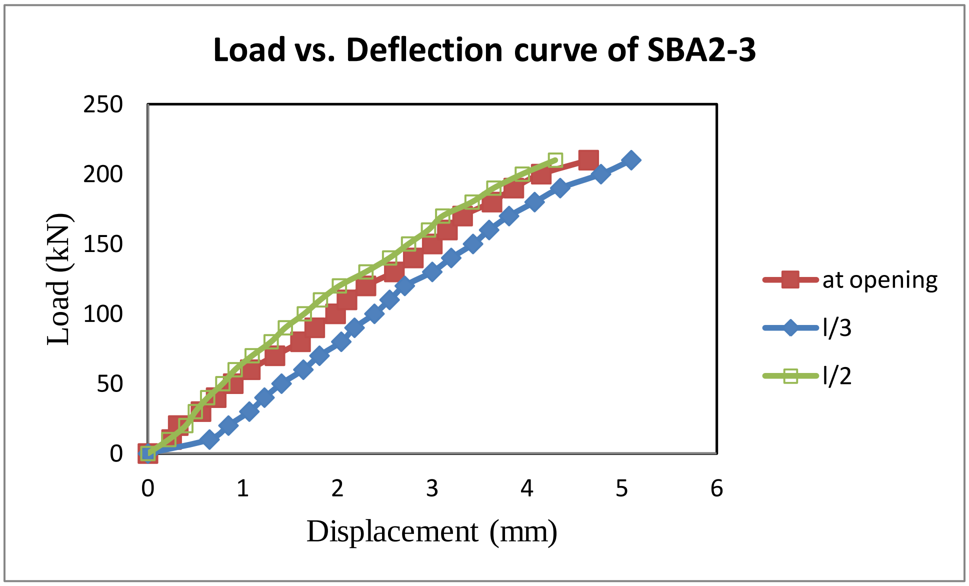


Figure 4-16. Load vs. Deflection Curve for SBA2-3

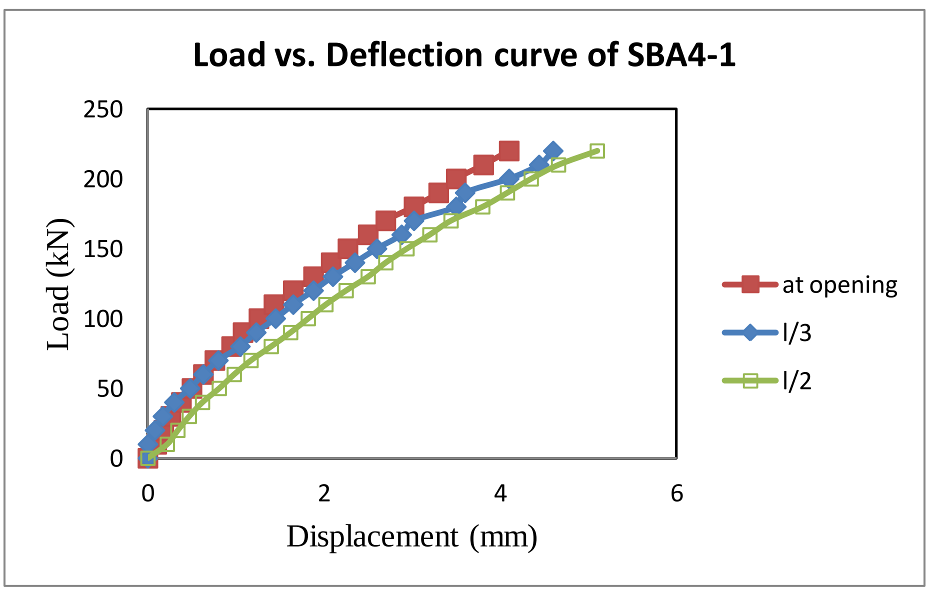


Figure 4-17. Load vs. Deflection Curve for SBA4-1

4.3.2. GROUP-B:

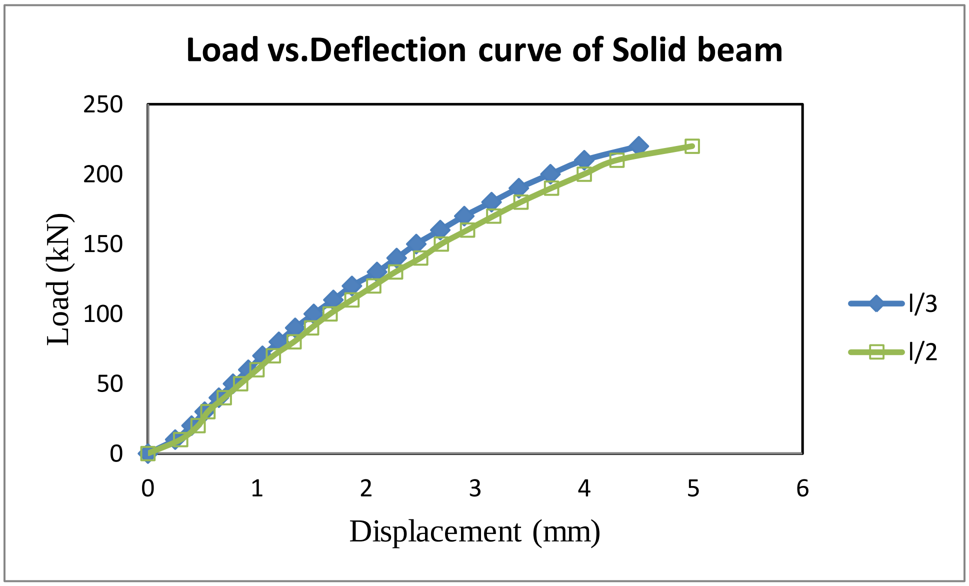


Figure 4-18. Load vs. Deflection Curve for Solid beam B

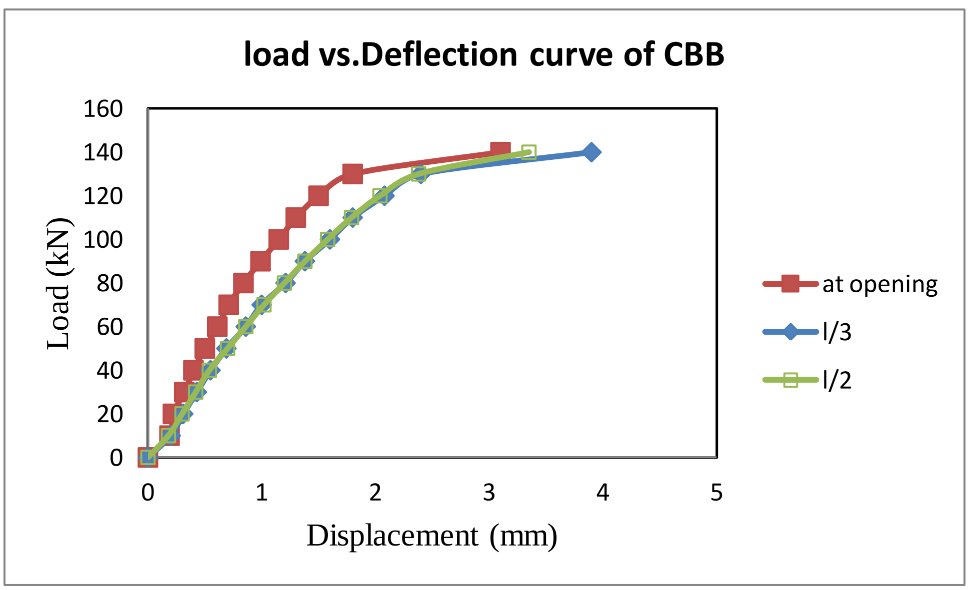


Figure 4-19. Load vs. Deflection Curve for CBB

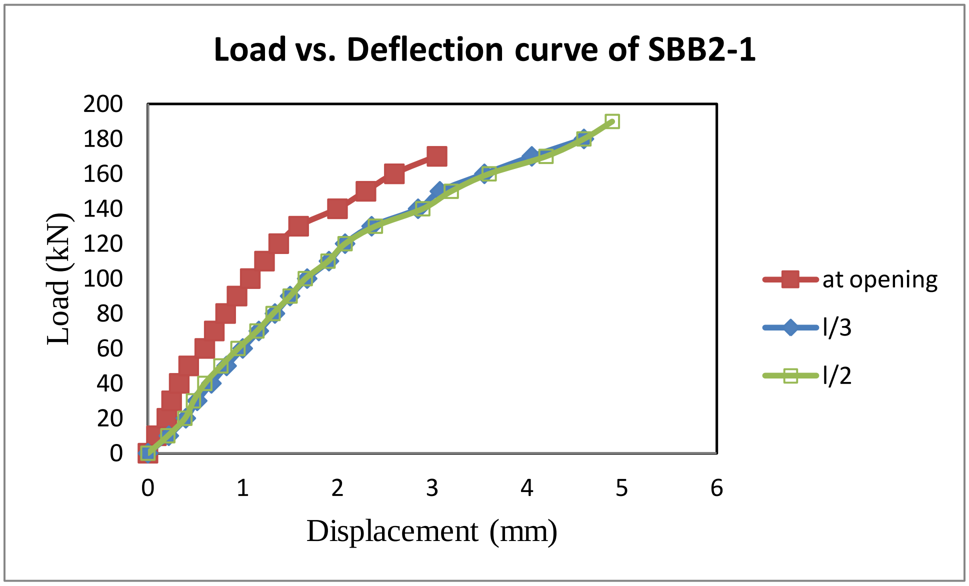


Figure 4-20. Load vs. Deflection Curve for SBB2-1

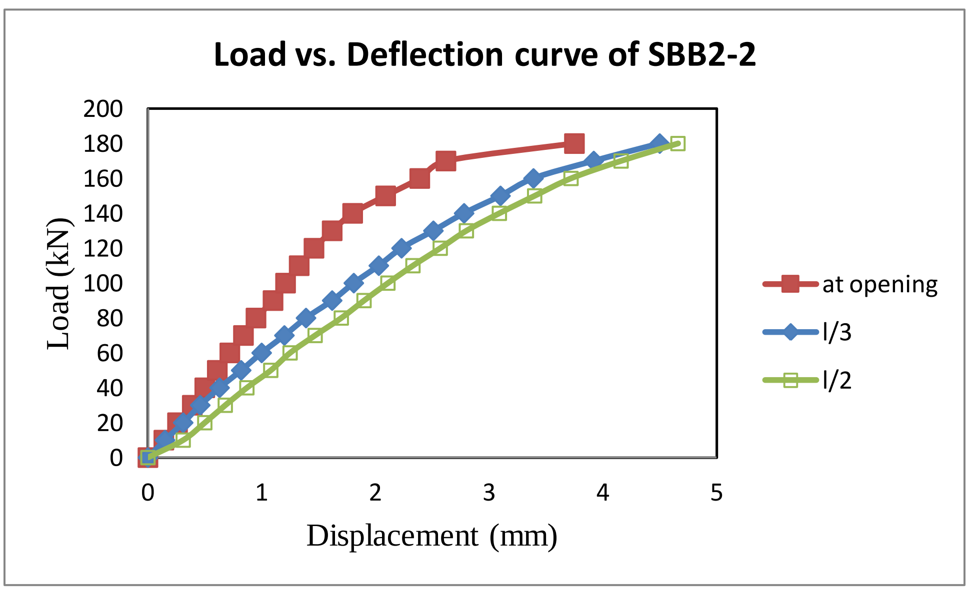


Figure 4-21. Load vs. Deflection Curve for SBB2-2

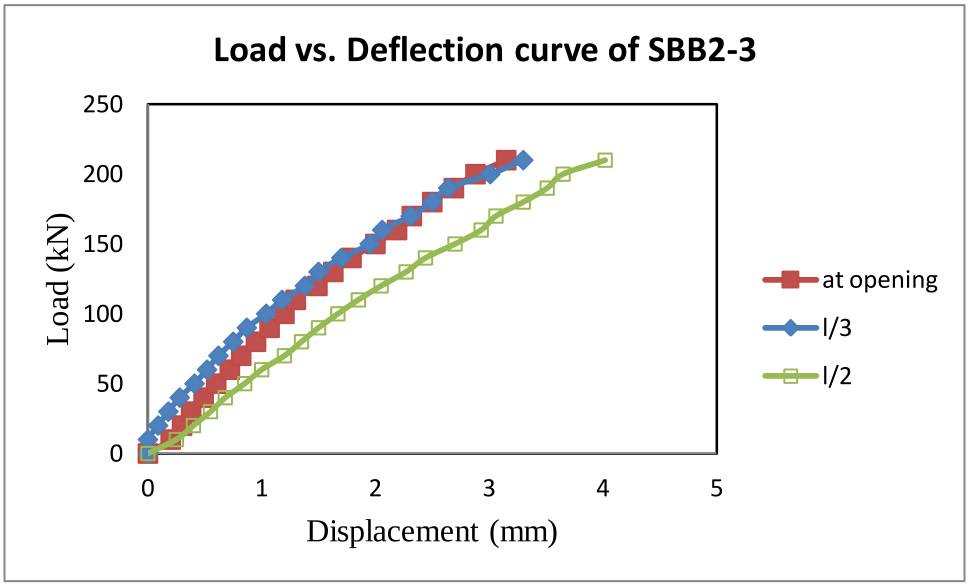


Figure 4-22. Load vs. Deflection Curve for SBB2-3

The deflection profile for the control beam CBA and beams SBA2-1 (strengthened with two layers continuous U-wrap on hole side), SBA2-2 (strengthened with two layers continuous U-wrap on both sides with flange anchorage system) and SBA4-1 (strengthened with four layers continuous U-wrap on both sides with flange anchorage system) are presented in figure 4-23. From the figure 4-23, it is observed that SBA2-2 and SBA4-1 performs well compared to CBA and SBA2-1. The reduction in mid-span deflection of the beam SBA4-1 compared to CBA and SBA2-1 are 20.39% and 31.91% respectively under the applied load of 160 kN.

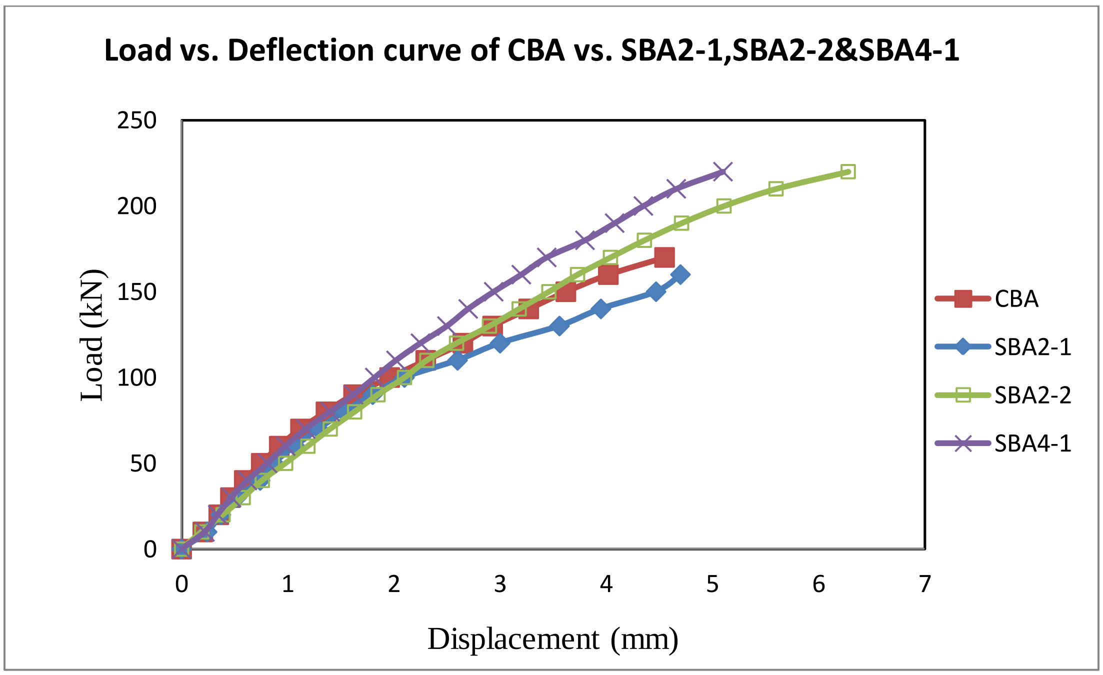


Figure 4-23. Load vs. Deflection Curve for CBA vs. SBA2-1, SBA2-2 and SBA4-1

The deflection profile for the control beam CBA and SBA2-3 (strengthened with two layers continuous U-wrap on both sides with flange anchorage system having a shear span of 250mm) are presented in figure 4-24. From the figure 4-24, it is observed that SBA2-3 performs well compared to CBA. The reduction in mid-span deflection of the beam SBA2-3 compared to CBA is 26.36% under the applied load of 160 kN.

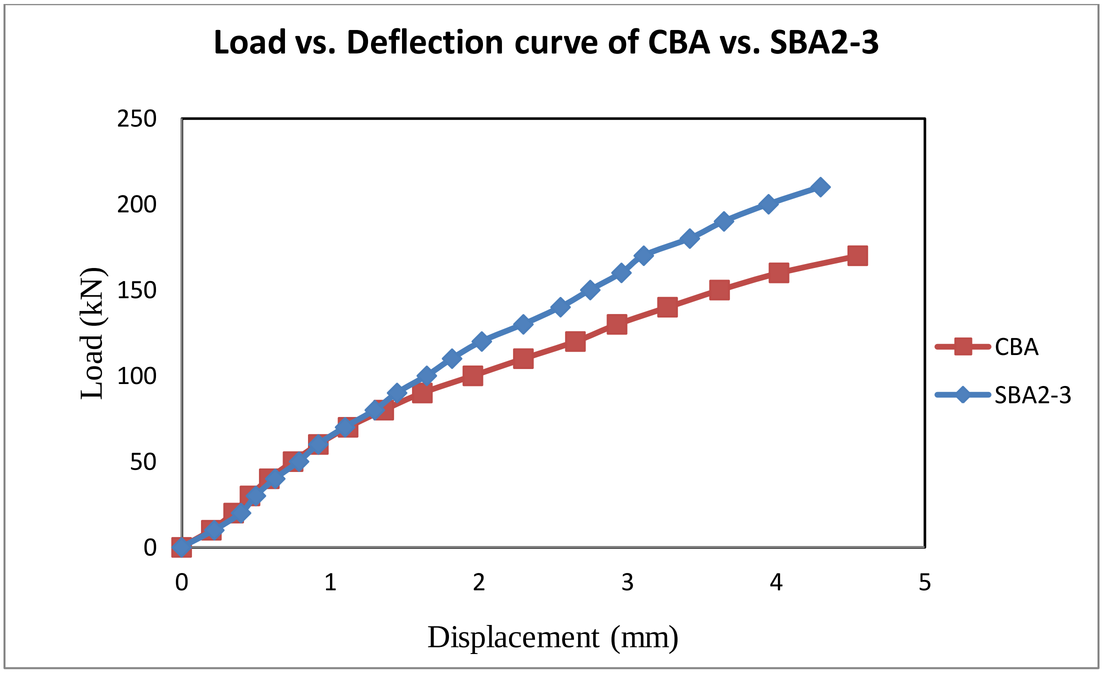


Figure 4-24. Load vs. Deflection Curve for CBA vs. SBA2-3

The deflection profile for the control beam CBA and solid beam (without transverse hole and no strengthening) are presented in figure 4-25. From the figure 4-25, it is observed that solid beam performs well compared to CBA. The reduction in mid-span deflection of the beam Solid beam compared to CBA is 10.44% under the applied load of 160 kN.

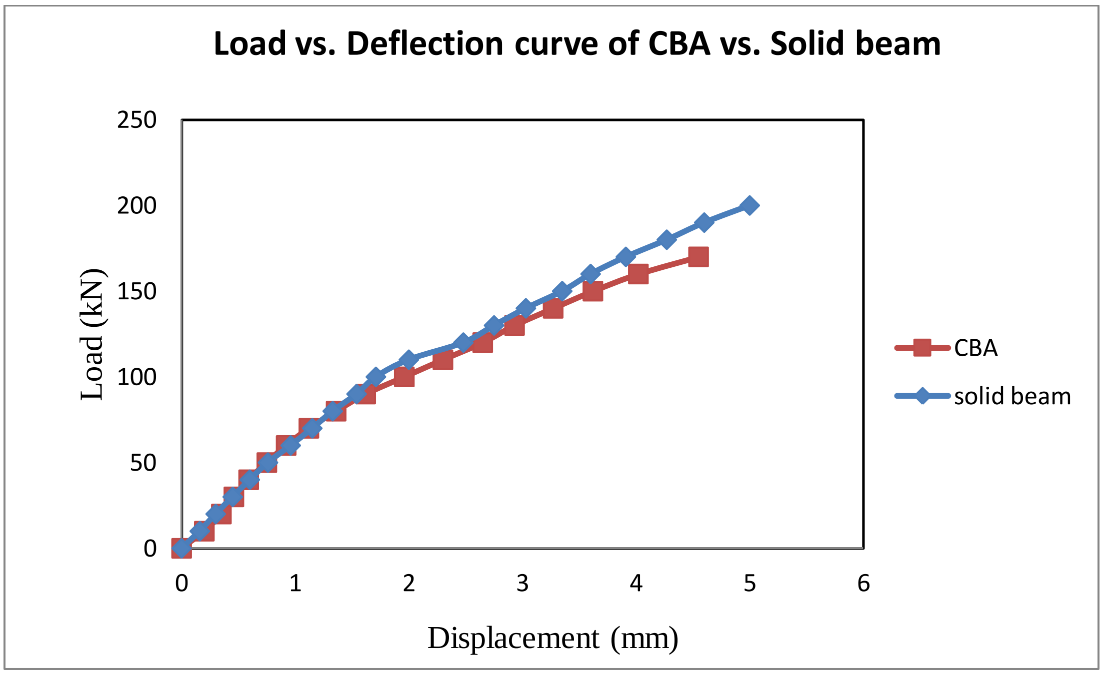


Figure 4-25. Load vs. Deflection Curve for CBA vs. Solid beam

The deflection profile for the control beam CBB and beams SBB2-1 (strengthened with two layers continuous U-wrap on hole side) and SBB2-2 (strengthened with two layers continuous U-wrap on both sides with flange anchorage system) are presented in figure 4-26. From the figure 4-26, it is observed that SBB2-1 and SBB2-2 performs well compared to CBB. The reduction in mid-span deflection of the beam SBB2-1 and SBB2-2 compared to CBB are 13.43% and 7.76% respectively under the applied load of 140 kN.

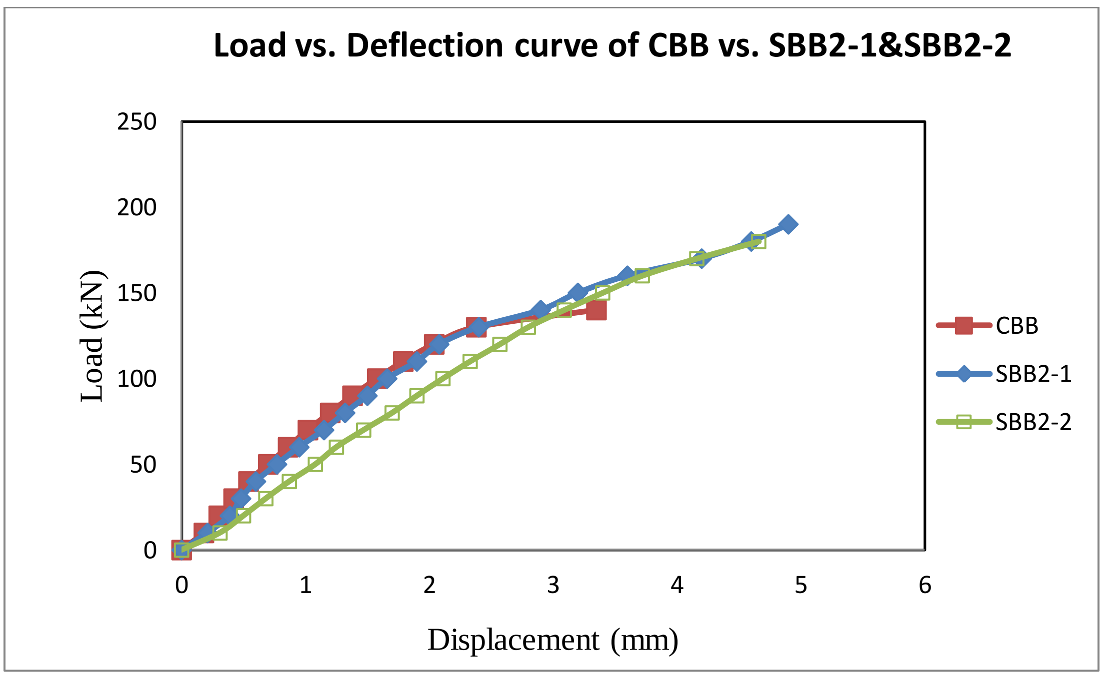


Figure 4-26. Load vs. Deflection Curve for CBB vs. SBB2-1 and SBB2-2

The deflection profile for the control beam CBB and SBB2-3 (strengthened with two layers continuous U-wrap on both sides with flange anchorage system having a shear span of 250mm) are presented in figure 4-27. From the figure 4-27, it is observed that SBB2-3 performs well compared to CBB. The reduction in mid-span deflection of the beam SBB2-3 compared to CBB is 27.16% under the applied load of 140 kN.

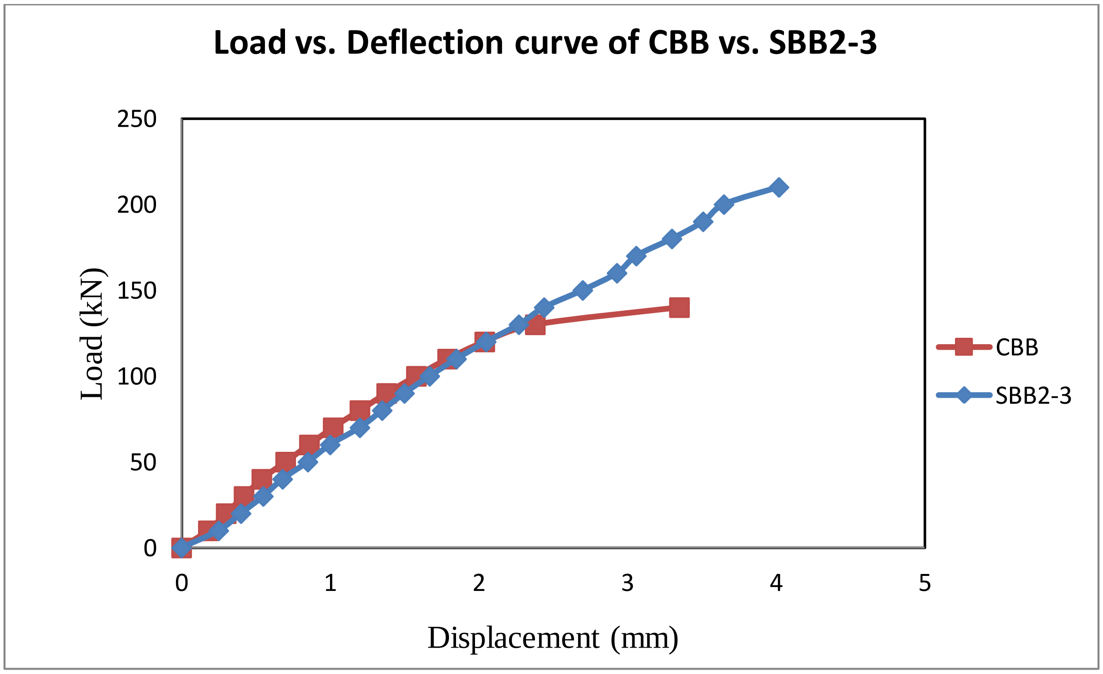


Figure 4-27. Load vs. Deflection Curve for CBB vs. SBB2-3

The deflection profile for the control beam CBB and solid beam (without transverse hole and no strengthening) are presented in figure 4-28. From the figure 4-28, it is observed that solid beam performs well compared to CBB. The reduction in mid-span deflection of the beam Solid beam compared to CBB is 25.37% under the applied load of 140 kN.

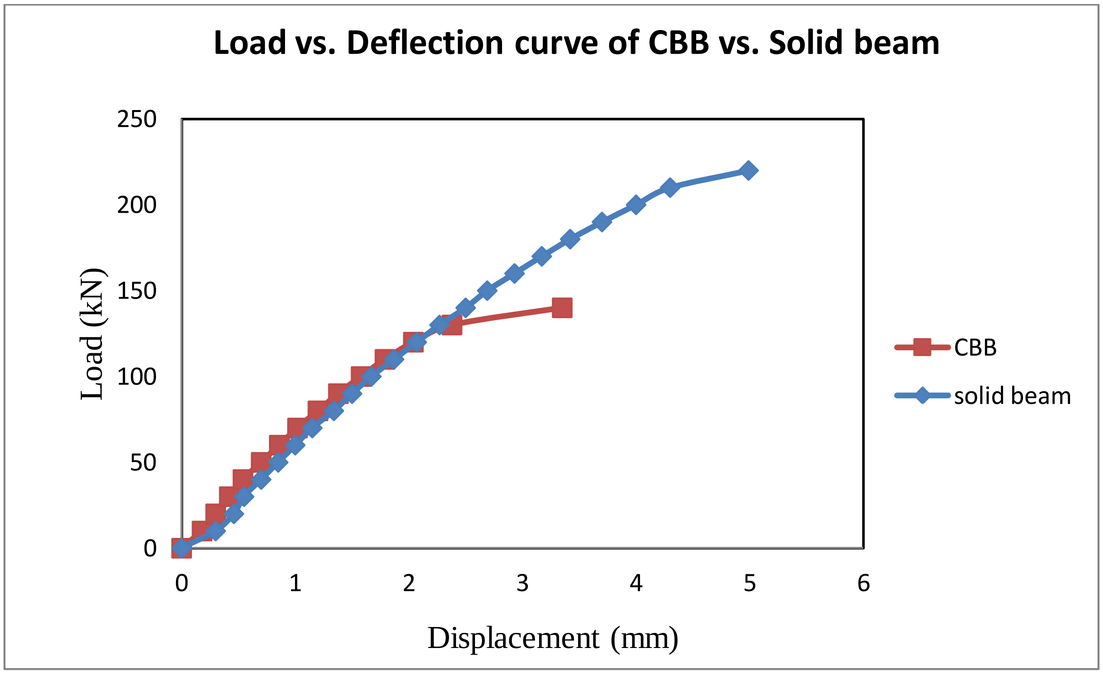


Figure 4-28. Load vs. Deflection Curve for CBB vs. Solid beam B

The ultimate load carrying capacities of all the beams along with the nature of failure are summarized in Table 4.1. The ratio of ultimate load carrying capacity of strengthened beam to control beam are computed and presented in Table 4.1.

Table 4.1 Ultimate load and nature of failure for various beams

Beam Designation		**Nature of Failure**	**P_u (kN)**	**$\lambda = \frac{Pu(\text{Strengthened Beam})}{Pu(\text{Control Beam})}$**
Group-A	**Solid beam**	Shear failure	208	1.2
	CB	Shear failure	172	-
	SB1	Shear failure	180	1.04
	SB2	Tearing and Debonding of GFRP + Shear failure	220	1.28
	SB3	Tearing and Debonding of GFRP + Shear failure	210	1.22
	SB4	Debonding failure + Shear failure	230	1.33
Group-B	**Solid beam**	Shear crack shifted to the non-strengthened zone of shear span	240	1.71
	CB	Shear failure	140	-
	SB1	Tearing and Debonding failure + Shear failure	198	1.41
	SB2	Tearing of GFRP + Shear failure	204	1.45
	SB3	Tearing of GFRP + Shear failure	214	1.53

CHAPTER – 5

THEORETICAL STUDY

CHAPTER - 5

THEORETECAL STUDY

5.1GENERAL

The design approach for computing the shear capacity of RC T-beams strengthened with externally bonded GFRP sheets is presented in this chapter. The design approach is expressed in American Concrete Institute (ACI) design code format. The main factors affecting the additional strength that may be achieved by the externally bonded GFRP reinforcement have been considered. The experimental model described two possible failure mechanisms of GFRP reinforcement such as GFRP debonding and GFRP rupture. The shear strength of Reinforced Concrete (RC) T-beams are theoretically computed for varying degree of FRP strengthening.

5.2 FACTORS AFFECTING THE SHEAR CONTRIBUTION OF FRP

Based upon the results of the experimental study, the contribution of externally bonded FRP to the shear capacity is influenced by the following parameters:

- Amount and distribution of FRP reinforcement
- Fiber orientation
- Wrapping schemes (U-wrap, or fiber attached on the two web sides of the beam)
- Presence of FRP end anchor
- Concrete surface preparation and surface roughness

5.3 SHEAR STRENGTH OF RC BEAMS STRENGTHENED WITH FRP REINFORCEMENT USING ACI CODE GIUDELINES

5.3.1 Design of Material Properties

The material properties reported by the manufacturers, such as the ultimate tensile strength, typically do not consider long-term exposure to environmental conditions and should be considered as initial properties. Because long-term exposure to various types of environments can reduce the tensile properties and creep-rupture and fatigue endurance of FRP laminates, the material properties used in design equations should be reduced based on the environmental exposure condition.

Eq.s (1) through (3) gives the tensile properties that should be used in all design equations. The design ultimate tensile strength should be determined using the environmental reduction factor given in the ACI 440.2R-02 document for the appropriate fiber type and exposure condition:

Design ultimate tensile strength = $f_{fu} = C_E f^*_{fu}$ (1)

where,

f_{fu} = design ultimate tensile strength of FRP,(MPa)

C_E = environmental reduction factor

f^*_{fu} = ultimate tensile strength of the FRP materials as reported by the manufacturer,(MPa)

Similarly, the design rupture strain should also be reduced for environmental-exposure conditions:

Design rupture strain = $\varepsilon_{fu} = C_E \varepsilon^*_{fu}$ (2)

where,

ε_{fu} = design rupture strain of FRP reinforcement,(mm/mm)

ε^*_{fu} = ultimate rupture strain of the FRP reinforcement,(mm/mm)

Because FRP materials are linearly elastic until failure, the design modulus of elasticity can then be determined from Hook's law. The expression for the modulus of elasticity, given in Eq. (3), recognizes that the modulus is typically unaffected by environmental conditions. The modulus given in this equation will be the same as the initial value reported by the manufacturer.

$$E_f = \frac{f_{fu}}{\varepsilon_{fu}} \quad (3)$$

The material used for this present work is glass fiber and epoxy resin, and the exposure condition is internal exposure. For present calculation the environmental reduction factor (C_E) is used as 0.75.

5.3.2 Nominal Shear Strength

The nominal shear strength of an RC beam may be computed by basic design equation presented in ACI 318-95 and given as in Eq. (4)

$$V_n = V_c + V_s \quad (4)$$

In this equation the nominal shear strength is the sum of the shear strength of the concrete (which for a cracked section is attributable to aggregate interlock, dowel action of the longitudinal reinforcement, and the diagonal tensile strength of the uncracked portion of the concrete) and the strength of the steel shear reinforcement.

In the case of beams strengthened with externally bonded FRP sheets, the nominal shear strength may be computed by the addition of a third term to account for the contribution of FRP sheet to the shear strength. This is expressed in Eq. (5)

$$V_n = V_c + V_s + V_f \tag{5}$$

5.3.3 Design Shear Strength

The design shear strength is calculated by multiplying the nominal shear strength by a strength reduction factor, ϕ. It is suggested that the reduction factor of ϕ = 0.85 given in code ACI 318-95 be maintained for the concrete and the steel terms.

The basic design equation for the shear capacity of a concrete member is;

$$V_u \leq \phi V_n \tag{6}$$

where,

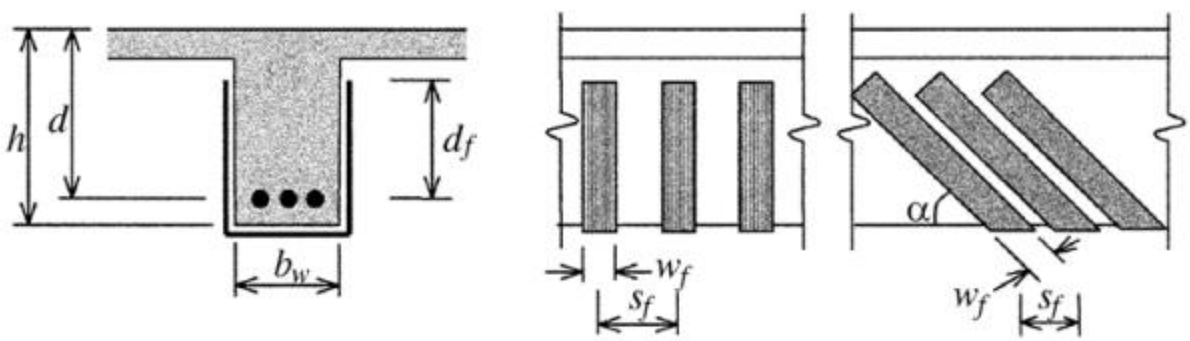
V_u is the total shear force applied at a given section due to the factored loads.

The nominal shear strength of an FRP-strengthened concrete member can be determined by adding the contribution of the FRP reinforcing to the contributions from the reinforcing steel (stirrups, ties, or spirals) and the concrete Eq. (7). An additional reduction factor ψ_f is applied to the contribution of the FRP system.

$$\phi V_n = \phi\,(V_c + V_s + \psi_f V_f) \tag{7}$$

It is suggested that an additional reduction factor ψ_f be applied to the shear contribution of the FRP reinforcement. For bond-critical shear reinforcement, an additional reduction factor of 0.85 (Completely wrapped members) is recommended. For contact-critical shear reinforcement, an additional reduction factor of 0.95 (Three-sided U-wraps or bonded face piles) is recommended in code ACI 440.2R-02.

5.3.4 FRP system contribution to shear strength



(a) (b) (c)

Figure - 5.1. Illustration of the dimensional variables used in shear-strengthening calculations for repair, retrofit, or strengthening using FRP laminates.

(a) Cross-section, (b) Vertical FRP strips, (c) Inclined FRP strips.

Figure – 5.1 illustrates the dimensional variables used in shear-strengthening calculations for FRP laminates. The contribution of the FRP system to shear strength of a member is based on the fiber orientation and an assumed crack pattern [Khalifa et al. 1998]. The shear strength provided by the FRP reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across the assumed crack. The shear contribution of the FRP shear reinforcement is then given by Eq. (8).

$$V_f = \frac{A_{fv} f_{fe}(sin\alpha + cos\alpha) d_f}{s_f} \qquad (8)$$

where,

In Eq. (8), A_{fv} is the area of one strip of transverse FRP reinforcement covering two sides of the beam. This area may be expressed as follows:

$$A_{fv} = 2\, t_f w_f \tag{9}$$

The area of GFRP shear reinforcement A_{fv} is the total thickness of the sheet (usually $2t_f$ for sheets on both sides of the beam) times the width of the GFRP sheet w_f in the longitudinal direction. The dimensions used to define the area of GFRP are shown in Figure 1. The spacing between the strips, s_f, is defined as the distance from the centerline of one strip to the centerline of an adjacent strip. For multilayered beam it is n times the area of GFRP shear reinforcement A_{fv} where, n is the number of layers. For the continuous vertical FRP reinforcement, the spacing of the strip, s_f, and the width of the strip, w_f, are equal. The angle α is angle between principal fiber orientation and longitudinal axis of the beam.

The other variable in Eq. (8), the tensile stress in the FRP shear reinforcement at ultimate stage, f_{fe} is directly proportional to the level of strain that is developed in the FRP shear reinforcement at ultimate as expressed in Eq. (10).

$$f_{fe} = \varepsilon_{fe} E_f \tag{10}$$

5.3.5 Effective strain in FRP laminates

The effective strain is the maximum strain that can be achieved in the FRP system at the ultimate load stage and is governed by the failure mode of the FRP system and the strengthened reinforced concrete member. All possible failure modes should be considered and the effective strain should be used which is the representative of the critical failure mode. The following subsections give guidance on determining this effective strain for different configurations of FRP laminates used for shear strengthening of reinforced concrete members.

Completely wrapped members:

For reinforced concrete beams completely wrapped by the FRP system, loss of aggregate interlock of the concrete has been observed to occur at fiber strains less than the ultimate fiber strain. To preclude this mode of failure, the maximum strain used for design should be limited to 0.4% for applications that can be completely wrapped with the FRP system as given in Eq. (11).

$$\varepsilon_{fe} = 0.004 \leq 0.75\ \varepsilon_{fu} \quad \text{(for completely warping around the members cross section)} \qquad (11)$$

This strain limitation is based on testing [Priestley et al. 1996] and experience. Higher strains should not be used for FRP shear-strengthening applications.

Bonded U-wraps or bonded face plies:

FRP systems that do not enclose the entire section (two- and three-sided wraps) have been observed to delaminate from the concrete before the loss of aggregate interlock of the section. For this reason, bond stresses should be analyzed to determine the usefulness of these systems and the effective strain level that can be achieved [Triantafillou 1998a]. The effective strain is calculated using a bond-reduction coefficient k_v applicable to shear.

$$\varepsilon_{fe} = k_v\,\varepsilon_{fu} \leq 0.004 \quad \text{(for bonded U-wraps or bonding to two sides)} \qquad (12)$$

where,

k_v = bond-reduction coefficient for shear.

5.3.6 Reduction coefficient based on Rupture failure mode

There is no particular guideline indicated for GFRP. The model proposed by Khalifa et al. (1998) is used to find out the reduction coefficient for rupture failure mode. The reduction coefficient presented as a function of $\rho_f E_f$ is shown in Equation (11) for $\rho_f E_f \leq 0.7$ GPa:

$$R = 0.5622\,(\rho_f E_f)^2 - 1.218\,(\rho_f E_f) + 0.778 \tag{11}$$

where,

ρ_f is the GFRP shear reinforcement ratio = $(2t_f/b_w)\,(w_f/s_f)$

E_f is the tensile modulus of elasticity of GFRP.

5.3.7 Reduction coefficient based on Debonding failure mode

The reduction coefficient based on debonding failure mode, is given in ACI 440.2R-02 design approach. k_v is used as bond reduction coefficient.

The bond-reduction coefficient is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the laminate. The bond-reduction coefficient can be computed from Eq. (13) through (16) [Khalifa et al. 1998].

$$k_v = \frac{k_1 k_2 L_e}{11{,}900\,\epsilon_{fu}} \leq 0.75 \tag{13}$$

The active bond length L_e is the length over which the majority of the bond stress is maintained.

This length is given by Eq. (14).

$$L_e = \frac{23{,}300}{(nt_f E_f)^{0.58}} \tag{14}$$

The bond-reduction coefficient also relies on two modification factors, k_1 and k_2, that account for the concrete strength and the type of wrapping scheme used, respectively. Expressions for these modification factors are given in Eq. (15) and (16).

$$k_1 = \left(\frac{f'_c}{27}\right)^{2/3} \tag{15}$$

$$k_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & for\ U - wraps \\ \dfrac{d_f - 2L_e}{d_f} & for\ two\ sides\ bonded \end{cases} \tag{16}$$

where,

f_c' is the concrete strength in MPa and

E_f is the tensile modulus of elasticity of GFRP in MPa.

5.4 Theoretical Calculations

The shear strength of the control beam and two strengthened beams (one failed by debonding while other failed by rupture) are theoretically computed and presented below.

Control Beam (CBB):

This is control beam is of group-B i.e shear reinforcement of 8mm φ at 200 mm spacing was provided.

The shear contribution of the concrete and steel are given below.

$$V_c = \frac{\sqrt{f_c{'}}}{6} b_w d = \frac{\sqrt{24.97}\ X\ 150\ X140}{6X1000} = 17.49 \text{ kN}$$

$$V_s = \frac{A_s\, f_{y\ d}}{s} = \frac{100.544\ X\ 415\ X140}{1000} = 29.21 \text{ kN (for no shear reinforcement in the beam)}$$

$V_n = V_c + V_s = 17.49 + 29.21 = 46.69$ kN

$\phi\ V_n = 0.85\ (46.69) = 39.69$ kN

Strengthened Beam 1 (SBB2-1):

The shear contribution of the concrete and steel are given below.

$$V_c = \frac{\sqrt{f_c{'}}}{6} b_w d = \frac{\sqrt{25.15}\ X\ 150\ X140}{6X1000} = 17.55 \text{ kN}$$

$$V_s = \frac{A_s\, f_{y\ d}}{s} = \frac{100.544\ X\ 415\ X140}{1000} = 29.208 \text{ kN (for no shear reinforcement in the beam)}$$

Shear contribution of the FRP:

Reduction coefficient for failure controlled by debonding

For continuous U-wrap with two layers,

$$\rho_f = \frac{2\ t_f}{b_w}\left(\frac{w_f}{s_f}\right)$$

For continuous vertical oriented ($\beta = 90^0$) GFRP, $w_f / s_f = 1$

$$\rho_f = \frac{2\ (1)}{150} = 0.0133$$

$E_f = 10.020$ GPa

$\rho_f E_f = 0.0133 \times 10.020 = 0.133$ GPa

$A_f = 2 \times t_f \times w_f = 2 \times 1 \times 1000 = 2000\ mm^2$

$$R = \frac{0.0042\,(f_c')^{\frac{2}{3}} w_{fe}}{t_f E_f^{\ 0.58} \in_{fu} d_f} = \frac{0.0042\,(25.25)^{\frac{2}{3}}\, X\, 90}{10.020^{0.58}\, X\, 0.0125\, X 140} = 0.487$$

$$R = \frac{\in_{fe}}{\in_{fu}}$$

Design ultimate strength (f_{fu})

$f_{fu} = C_E\ f_{fu}^{\ *}$

C_E = Environmental reduction factor = 0.75 (for exterior condition , Glass and epoxy)

f_{fu} = 0.75 x 167.7 = 125.75 MPa

$\in_{fu}^{\ *} = \frac{f_{fu}^{\ *}}{E_f} = \frac{167.7}{6.829} = 0.0167$

$\in_{fu} = C_E$ x $\in_{fu}^{\ *}$ = 0.75 x 0.0167 = 0.0125

$\in_{fe}$ = R x $\in_{fu}$ = 0.487 x 0.0125 = 0.0061

$\in_{fe} \leq 0.004$ (As per ACI 440.2R)

So used $\in_{fe}$ = 0.004

$f_{fe} = \in_{fe}$ x E_f = 0.004 x 10.020 =40 MPa

$V_f = \frac{A_{fv} f_{fe}(sin\alpha + cos\alpha) d_f}{s_f} = \frac{2000\ X\ 40\ X\ 140}{1000}$ = 11.2 kN

$\phi\ V_n$ = 0.85 (17.55+29.208) +0.7(11.2) = 39.74 + 7.84 = 47.58 kN

The design shear strength of the remaining beams can be computed in similar way. The nominal and design strength of all the strengthened and control beams are tabulated in Table 5.1 along with the experimental results.

5.5 Comparison of Experimental Results with ACI prediction

The shear strength of the beams strengthened with GFRP sheets obtained from the experimental study is compared to the design shear strength predicted by the ACI code (ACI 440.2R-02) guidelines. Different nomenclatures used in Table 5.1 are explained below for clarity.

$V_{n,test}$ = Total nominal shear strength by test,

$V_{c,test}$ = nominal shear strength provided by concrete obtained from test,

$V_{s,test}$ = nominal shear strength provided by steel shear reinforcement obtained from test,

$V_{f,test}$ = nominal shear strength provided by shear reinforcement obtained from test,

$V_{n,theor}$ = nominal shear strength calculated theoretically using ACI guidelines,

$V_{c,theor}$ = nominal shear strength provided by concrete theoretically,

$V_{s,theor}$ = nominal shear strength provided by steel shear reinforcement theoretically,

$V_{f,theor}$ = nominal shear strength provided by GFRP shear reinforcement theoretically.

Table 5.1 Comparisons of experimental and ACI predicted shear strength results

Speci-men	Experimental Results						Theoretical results predicted by ACI 440.2R-02 Design approach			
	Load at failure	$V_{n,test}$ (kN)	$V_{c,test}$ (kN)	$V_{s,test}$ (kN)	$V_{f,test}$ (kN)	$V_{f,test}/V_{n,test})*100$ (%)	$V_{f,theor}$ (kN)	$V_{c,theor}$ (kN)	$V_{s,theor}$ (kN)	$\phi V_{n,theor}$ (kN)
Solid beam	208	104	104	-	-	-	-	17.55	0	14.91
CBA	172	86	86	0	-	-	-	16.01	0	13.6
SBA2-1	180	90	86	0	4	4.44	11.2	16.36	0	21.74
SBA2-2	220	110	86	0	24	21.81	11.2	17.05	0	22.33
SBA2-3	210	105	86	0	19	18.09	11.2	15.83	0	21.29
SBA4-1	230	115	86	0	29	25.21	17.89	18.12	0	27.92
Solid beam	240	136	106.8	29.2	-	-	-	16.06	29.21	38.48
CBB	140	70	48.8	29.2	-	-	-	16.69	29.21	14.18
SBB2-1	198	99	48.8	29.2	21	21.21	11.2	15.94	29.21	46.21
SBB2-2	184	92	48.8	29.2	14	15.21	11.2	16.96	29.21	47.08
SBB2-3	214	107	48.8	29.2	29	27.1	11.2	17.45	29.21	47.5

It is observed from the Table 5.1 that the ACI prediction give satisfactory and conservative results when compared to that of experimental results for the all strengthened beams.

Table 5.2 Comparison of shear contribution of GFRP sheet from experimental and ACI Guidelines

Specimen Designation	Experimental Results	Results as per ACI Guideline	
	$V_{f,test}$ *(kN)*	$V_{f,theor}$ *(kN)*	$V_{f,test}/V_{f,theor}$
SBA2-1	4	11.2	0.357
SBA2-2	24	11.2	2.14
SBA2-3	19	11.2	1.69
SBA4-1	29	17.89	1.62
SBB2-1	21	11.2	1.875
SBB2-2	14	11.2	1.25
SBB2-3	29	11.2	2.59

It is found from the Table 5.2 that the ratio of $V_{f,test}$ to $V_{f,theor}$ is the highest for the beam SBB2-3 strengthened with continuous FRP U-wrap with flange anchorage system for a shear span of 250 mm and lowest for the beam SBA2-1 strengthened with continuous FRP U-wrap on hole side.

CHAPTER – 6

CONCLUSIONS

CHAPTER - 6

CONCLUSIONS

In this experimental investigation the shear behaviour of RC T-beams strengthened by GFRP sheets are studied. The test results illustrated in the present study showed that the external strengthening with GFRP composites can be used to increase the shear capacity of RC T-beams, but the efficiency varies depending on the test variables such as fiber orientations, wrapping schemes, number of layers and anchorage scheme.

Based on the experimental and theoretical results, the following conclusions are drawn:

- The test results confirm that the strengthening technique of FRP system is applicable and can increase the shear capacity of T-beams.
- The experimental verification of the flange anchorage system shows the effectiveness in increasing the shear capacity of RC beams.
- Existing evidence clearly indicates that the anchorage system can make FRP strengthening even more attractive and economical for concrete repair and strengthening.
- The test results indicates that the contribution of GFRP benefits the shear capacity to a greater degree for beams without steel shear reinforcement than for beams with adequate steel shear reinforcement.
- The contribution of externally bonded GFRP reinforcement to the shear capacity is influenced by the shear span-to-depth ratio (a/d) and it increases with a decrease in a/d ratio.

- The use of anchorage system eliminates the debonding of the GFRP sheet, and consequently results in a better utilization of the full capacity of the GFRP sheet.

CHAPTER – 7

REFERENCES

CHAPTER - 7

REFERENCES

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